BY HORACE R. THAYER

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VOLUME II

DESIGN OF SIMPLE STRUCTURES

BY

HORACE R. THAYER

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CARNEGIE INSTITUTE OF TECHNOLOGY

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PREFACE

THIS work presupposes a knowledge of mechanics, stresses, and the mathematics on which they depend. While it also assumes that the reader has Vol. I, its independent use has been carefully considered.

We discuss here the design of such simple structures as beams, girders, viaducts, trusses, buildings, stand-pipes, and elevated tanks. An attempt has been made to approach these subjects from both theoretical and practical standpoints and to give many concrete examples. A decided appreciation of the magnitude of the task has come with the progress of the work. However, it is hoped that the references which have been compiled and arranged with a view to access and availability may supply deficiencies to some extent.

The author takes this occasion to acknowledge his indebtedness to many sources of information These in general are represented by the references. The writer feels particular obligations to our high-grade technical publications. Many thanks are also due the reviewers; Messrs. C. M. Canady and E. P. Dudley, engineers for American Bridge Co.; Mr. P. M. Deforth, designing engineer for the Jones and Laughlin Steel Co.; Mr. J. L. Latham, detailer for McClintic-Marshall Construction Co.; Mr. S. M. Siesel, assistant engineer for the J. L. Stuart Co. Messrs. W. E. Culp and P. J. DeMuth of the Senior Class in Civil Engineering at the Carnegie Institute of Technology have read the manuscript. Clarence and Richard Thayer have assisted in correcting and revising proofs. The aid rendered by various manufacturers and fabricators is

acknowledged in the text. The list would be incomplete without mentioning the facilities and courtesies extended by the Carnegie Library of Pittsburgh.

The author wishes to thank the public for their appreciation of his first work and the reviewers in the technical press for their fairness.

Additional information, criticism, and suggestion are always welcome.

H. R. T.

Carnegie Institute of Technology,
Pittsburgh, Pa., May 7, 1914.

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CHAPTER VI

BEAM BRIDGES—PLATE GIRDERS

Art. 70. General

A *beam bridge* is one where the loads are carried to the supports by beams. These may be rolled or built up. We shall here consider only those bridges which are used for railroads, electric cars, and highways.

FIG. 70*a*.—Deck Plate Girder Bridge Carrying Midland Division of New York, New Haven & Hartford Railroad over Blackstone River, Blackstone, Mass.

We may classify these bridges as *deck*, *through*, and *half through*. In the former, the floor is laid on top of the beams. Fig. 70*b* shows a deck plate girder for a railroad span. Rails rest on ties, which in turn are supported by the built beams; these are carried at either end by the shoes which transmit

the reactions to the abutments. There should be three systems of bracing, one each in the plane of the top and bottom, known as the upper and lower lateral systems, and sway bracing in planes perpendicular to the beams.

In a *through bridge*, the floor comes at or near its bottom flange. Fig. 70*c* represents section for a case of this kind,

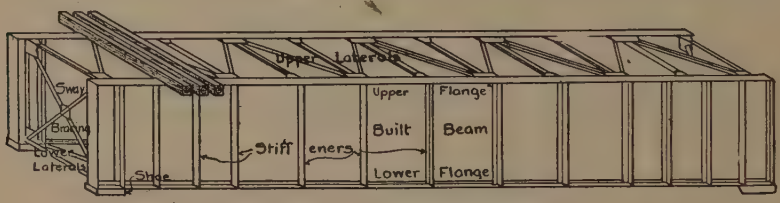


FIG. 70*b*.—A Deck Plate Girder.

a solid floor bridge for a railroad. The "gussets" take the place of the sway bracing in the deck structure. Lower lateral system may be omitted if floor furnishes sufficient bracing. Sometimes the ties rest directly on the bottom chord, but this type is nearly obsolete on account of the difficulty of properly bracing top of girder and the objectionable combination of stresses in the lower flange. See Art. 76.

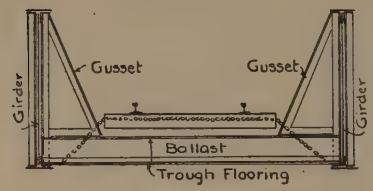


FIG. 70*c*.

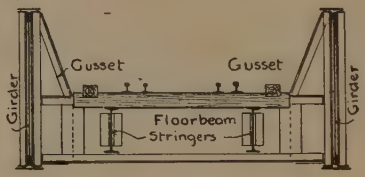


FIG. 70*d*.

Cross Sections of
Through Plate Girder Bridge. Half Through Plate Girder Bridge.

When the floor is part way between top and bottom flanges, the structure is termed "half through." Fig. 70*d* shows a section of such a railroad bridge. Like the preceding type, the upper laterals must be omitted, the sway bracing replaced by gussets, and the lower laterals inserted, unless a solid floor is used. This is not common for half through railroad bridges.

Considering the bridge alone, the *deck* type is best. It

is stiffer, cheaper, and safer. Its principal disadvantage is in locations where the higher elevation of floor increases cost of approaches, Fig. 70*f*. Here the cost of maintenance and repair, while somewhat greater for the through structure,

FIG. 70*e*.—Half Through Plate Girder Bridge of the C. H. & D. Ry. at Hamilton, Ohio. McClintic-Marshall Construction Company, Rankin, Pa.

may safely be neglected in comparative estimates. The important items are interest on the first cost of improvement and operating expenses of railroad traffic. These should be carefully studied to find best designs.

FIG. 70*f*.—Sketch Showing Relative Economy of Deck and Through Railroad Bridges for Abolishing a Grade Crossing.

The advantages of beam bridges are:

(*1*) Simplicity.
(*2*) Lack of adjustment.
(*3*) Freedom from accidents.
(*4*) Rigidity.
(*5*) Ease of erection.
(*6*) Economical for short spans.

Their disadvantages are:

(7) Appearance.

(8) Difficulty in shipping long spans.

Up to 80 feet in length, beam bridges should be used, except perhaps where teaming to site is necessary. Above this limit, trusses are likely to weigh less than plate girders, the difference increasing with the span. This economy is due to the greater depth of the former and the better distribution of its web material. Owing to the increased shop work, the cost per pound of the truss is more and it is not generally economical for spans less than one hundred feet. This is the usual upper limit for plate girders, but they are often employed as long as customary shipping facilities will permit, about 130 feet, on account of the above named advantages.

We will now take up the design of beam bridges, simply naming a few of the more important principles, referring to Chapters IV and V of Vol. I for further information.

(1) *Safety*

(*a*) Not less than two complete systems of bracing must be provided. These should suffice, not only to carry any transverse external loads, but in addition hold the girders to their duty.

(*b*) The compression flange is a column, and, if unsupported for such a length as to weaken it, allowable stress must be reduced. (Art. 50, *d* 11.)

(*c*) Examine shearing stresses in webs. (Art. 54, *a*.) Although often omitted in practice for rolled section, it is quite likely to determine strength of a short heavily loaded beam.

(*d*) Check computations after details are made to be sure that net area is at least equal to that assumed. (Art. 50, *d* 7.)

(*e*) So design the flange that long rivets are not necessary. (Art. 47 (2)).

(2) *Durability*

(*a*) Use no material less than $\frac{3}{8}$ inch thick for members carrying stress in a railroad structure, nor less than $\frac{5}{16}$ inch for highway. Since corrosion acts on the surface, the percent-

age lost in a given time is much greater for thin material.

(*b*) Keep all parts accessible that rusting may be prevented by painting, and in order to allow of inspection.

(*c*) Avoid pockets which would hold water.

(*d*) Design with allowable stresses which are low enough to prevent the rapid deterioration of the metal. It might seem to the student that 10,000 pounds per square inch, the customary allowable stress in railroad bridges, is lower than necessary in view of the fact that the elastic limit is not less than 30,000. However, we must allow for rusting, unequal distribution of stresses, approximate assumptions, and blows and shocks caused by any slight unevenness in the track.

(*3*) *Economy*

(*a*) In material:

In built beams, make weight of both flanges equal to that of the web and its fittings, Art. 52 (*1*).

Keep flange area as far from the neutral axis as is consistent with the other requirements.

Allow low stresses but keep as close to this limit as possible, making strength uniform throughout. The capacity of a chain is not altered by the addition of a few strong links.

Stagger rivets at points where section is fully stressed.

(*b*) In shopwork:

Keep rivets the same size if possible, Art. 47 (*4*).

Arrange girder and its flanges to be "racked," Art. 40.

Endeavor to so space rivets that all may be driven by machine.

Do not use material thicker than the diameter of the rivet, Art. 44 (*2*).

(*c*) In shipment, Art. 48.

(*d*) In erection, Art. 49.

REFERENCES

"Types and Details of Bridge Construction, Part II, Plate Girders," by Skinner.

"Design of Plate Girders," by Moore.

"Roofs and Bridges, Part III, Bridge Design," by Merriman and Jacoby.

"Theory and Practice of Modern Framed Structures," by Johnson, Bryan, and Turneaure.

"Notes on Plate Girder Design," by Hudson.

"Design of Plate Girders," International Library of Technology.

"Engineering Record," Vols. 49, 50 and 51, series by Skinner.

Prob. 70. Tyrrell gives for total weight of steel in typical single-track plate girders:

$$\text{Deck, } 100L+9L^2. \quad \text{Half through, } 300L+12L^2,$$

where L is span in feet and results are in pounds. Let Fig. 70*f* represent a proposed elimination of a crossing over a 50-foot highway, with a single-track railroad whose grade was originally level. If distance between street and base of rail is 23 feet and 18 feet respectively for the two schemes, compare cost of fill and bridge. Maximum grade 1%; steel 4¢ per pound, fill 25¢ per cubic yard.

Art. 71. I-Beam Bridges for Railroads

The weight in pounds per foot of an I beam to carry a given moment, M, at a stated fiber stress, S, is, roughly,

$$w=7\sqrt{M/S},$$

the last two quantities being in pounds and inches.

Using approximate formulæ, the areas of flange and web of a plate girder are respectively,

$$a=M/Sh \quad \text{and} \quad th,$$

where h equals height and t thickness of web, both in inches. The former should be multiplied by two to take care of both flanges and 10% added to allow for rivet heads and the deductions for net area. The customary thickness of the web is

$\frac{3}{8}''$ and we must add about 85% for stiffeners, fillers, and rivet heads. The total cross-sectional area then becomes,

$$a = 2.2M/Sh + 0.70\,h \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

As proven in Art. 52,

$$2.2\,M/Sh = 0.70\,h \quad \text{or} \quad h = 1.75\sqrt{M/S} \quad . \quad . \quad . \quad (2)$$

Substituting for h in (1),

$$a = 2.5\,\sqrt{M/S} \text{ or weight in lbs. per lin.ft. is } 9.0\sqrt{M/S}.$$

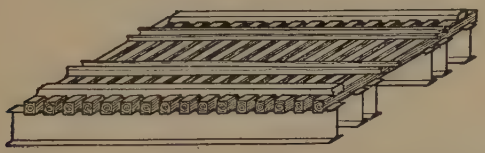


FIG. 71*a*.—Isometric of an I-Beam Railroad Bridge.

The I-beam span will be much less expensive for shop work. Allowing 3 and $3\frac{1}{2}$ cents for the total cost per pound erected of rolled and built I's respectively, we have to carry any given moment, M:

Cost of built beam in cents per lineal foot, $32\sqrt{M/S}$.

Cost of single I-beam in cents per lineal foot, $21\sqrt{M/S}$.

Cost of double I-beam in cents per lineal foot, $30\sqrt{M/S}$.

Cost of triple I-beam in cents per lineal foot, $36\sqrt{M/S}$.

We find then that for spans requiring more than two I-beams on each side, a plate girder should be used. For a uniform load of 5000 pounds per lineal foot per rail, about the equivalent of Cooper's E50, two I's, 24″ @ 100# will suffice for spans of 23 feet if allowable stress be taken at 10,000 lbs. per square inch. By using three beams, we may extend limit to 28 feet. Employing the Bethlehem 30″ I at 200#, we find these limits raised to 40 and 50 feet respectively. Present practice seems to favor I-beam bridges up to about 25 feet. With the Bethlehem

sections, this might well be increased to perhaps 35 feet. Depth should not be less than $\frac{1}{15}$ of the span for railroads and $\frac{1}{30}$ for highways.

I-beam bridges are seldom other than deck structures. There are two principal types, (*1*) *open floors* and (*2*) *solid floors*. The latter are demanded for bridges crossing city streets to prevent the dripping of oil and so forth from the trains. They are easy riding and cheap to maintain, but so expensive in first cost that they are advisable only when necessary for first-named reason. The amount of steel in a solid through floor

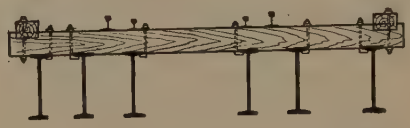


FIG. 71*b*.—Bridge Deck Construction.

will vary somewhat. It may be expressed roughly by the formula:

$$\text{Weight in lbs. per sq.ft.} = 10\sqrt{Pl/S},$$

where P is the maximum wheel load in pounds per rail;

l, the distance center to center of girders in feet;

S, the allowable unit stress in pounds per square inch.

The cost of maintenance of track may, for estimating purposes, be taken as 10 cents per foot per track per year for solid floor, and three times that amount for the open floor.

OPEN-FLOOR BRIDGES.*

The rails, both main and guard, rest upon ties. Usual spacing of former is 4′ $8\frac{1}{2}$″ inside to inside.† The ties should be not less than 8″×8″×9 to 12 feet, spaced 6″ apart in the clear. Figure each to carry one-third the maximum wheel load, using values given for bridges in Art. 6, except that shearing values may be doubled. Yellow pine or white oak are preferred. Every third tie should be fastened by $\frac{3}{4}$″ hook bolts to stringers. Ties should be notched $\frac{1}{2}$″ over support,

* Engineering News, Vol. 62, p. 270.

† This is termed the "gage."

as shown in Fig. 71*b*. In bridges built on a curve, the necessary super-elevation can be obtained by the use of the tapered tie, Fig. 71*c*, or by a cushion block, Fig. 71*d*. Guard rails should be placed 10″ inside, c. to c. of main rails. Guard timbers are about 8″ horizontal by 6″ vertical, the latter notched to 5″ over ties and bolted to them by $\frac{3}{4}$″ bolts at every splice and third tie. They are placed about 20″ outside of main rails, c. to c.

Fig. 71*c*.

Fig. 71*d*.

Bridge Deck Construction on Curves.

If one I beam is used under each rail, they should be placed about 6′ 6″ apart. This distance gives an elastic track, provides some margin of safety in case of derailment, and does not unduly stress the ties. When two or three I-beams are employed, sufficient space should be left between them for driving rivets, inspection, and painting. Sometimes separators, Fig. 137*f*, are used and the beams placed about as close as the flanges permit. This seems, however, like poor design. Preferably 5″ in the clear should be left between the flanges. Arrange to distribute the load as equally as possible between the beams. Figs. 71*e* and 71*f* are bad in this respect. On the other hand, Figs. 71*g* and 71*h* may be criticised as affording less elasticity and stability in case of derailment. The latter objection may be overcome, at a considerable expense, by putting a safety stringer under guard timber.

It is necessary that beams be braced at ends to hold them vertical under their loads. We may use:

(1) *X bracing*, Fig. 71*i*.

(2) *Solid bracing*, usually a channel, rolled or built, Fig. 71*j*.

(3) *Knee bracing* or *gussets*, Fig. 71*k*.

(4) *Separators*. These, as already mentioned, are objectionable for this location.

Support is also necessary for flanges at intervals not greater than fifteen times width of top flange, Art. 50, *d 11*.

The horizontal wind load against train is usually taken at 30 lbs. per sq.ft. times 10 feet high equals 300 lbs. per lin.ft. Adding 100 lbs. for the beams, we have 400 lbs. per lin.ft. acting horizontally and at right angles to the beams. The

Fig. 71*e*. Fig. 71*f*.

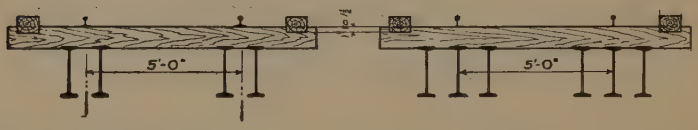


Fig. 71*g*. Fig. 71*h*.

Arrangement of Stringers.

guard timbers, taken as $4'' \times 8''$ net, will carry this load for spans of 10 feet or less. Greater lengths ought to have lateral bracing. However, it is sometimes omitted. It will be found to cause additional stresses of about 40%, increasing somewhat with the span, if we neglect guard timbers. This is not the best practice (Art. 50, *c 2*), but a bridge so designed would not thereby be rendered unsafe.

In a bridge 30 feet long, the end shear due to a wind load of

Fig. 71*i*. Fig. 71*j*. Fig. 71*k*.

Methods of Transverse Bracing.

400 lbs. per lin.ft. would be 6000#. Minimum allowable sized angle, a $3'' \times 3'' \times \frac{3}{8}''$ with three rivets at either end will carry this stress unless the length is permitted to become so great that allowable compressive unit stress is low. Fig. 71*l* shows

method of arranging bracing. There is a slight preference for an odd number of panels, since it makes the two beams alike.

At each corner there are two plates —a sole plate countersunk-riveted to the I-beams and a masonry plate. Expansion may be provided for at one end by slotting holes. See Types *1* and *2*, Art. 61.

FIG. 71*l*.—Arrangement of Laterals.

COMPUTATION OF AN I-BEAM BRIDGE

Let it be required to design a medium steel I-beam span of 20 feet for Cooper's E50. Allowable stresses as follows:

Flexure	17,000	lbs. per sq.in.
Shear	11,000	"
Bearing on masonry	400	"

Allowance for impact in percentage, (*C*) Art. 88, $\frac{30{,}000}{L+300}$

Weight of track, 200 lbs. per foot per rail.

The equivalent uniform load for 20 feet is 5200# per ft. per rail; estimated weight of each of the two beams assumed to be on each side, is then, taking allowable stress at 9000,

$$w = 7\sqrt{\frac{M}{S}} = 7\sqrt{\frac{1{,}560{,}000}{9000}} = 93, \text{ say } 100 \text{ lbs. per lin.ft.}$$

The dead load moment for each beam then becomes,

$$200 \cdot 20 \cdot 20 \cdot 12/8 = 120{,}000 \text{ in.lbs.}$$

The maximum live load moment is obtained by placing in position for *absolute maximum moment*, Fig. 71*m*. See Art. 85.

$$M = 12\left(\frac{50{,}000 \cdot 8.75 \cdot 8.75}{20} - 12{,}500 \cdot 5\right) = 1{,}547{,}000 \text{ in.lbs.}$$

Allow for impact, $1{,}547{,}000 \cdot \frac{300}{320} = 1{,}450{,}000$ in.lbs.

$$120{,}000 + 1{,}547{,}000 + 1{,}450{,}000 = 3{,}117{,}000 \text{ in.lbs.}$$

$$3{,}117{,}000/17{,}000 = 183.4 = I/c \text{ required in inches.}$$

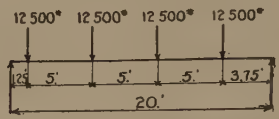


FIG. 71*m*.—Position for Absolute Maximum Moment.

Use two I's, 24″ @ 90# under each rail.
Maximum shear per rail equals,

$$\frac{400 \cdot 20}{2} + 25{,}000\left(1 + \frac{15}{20} + \frac{10}{20} + \frac{5}{20}\right)\left(1 + \frac{300}{320}\right) = 125{,}100 \text{ lbs.}$$

Shearing unit stress equals,

$$125{,}100/2 \cdot 24 \cdot 0.63 = 4170 \text{ lbs. per sq.in.} \quad \text{O. K.}$$

While actual reaction might be a little larger, we shall consider it equal to the end shear.

Bearing area required on concrete $= 125{,}100/400 = 313$ sq. inches.

We will not consider impact in designing anchor bolts.* Here we may have a sidewise thrust, carried by 4 bolts, of

$$400 \cdot 20/2 = 4000 \text{ lbs.,}$$

or a longitudinal force of,

$$200{,}000 \cdot 0.20 = 40{,}000 \text{ lbs.}$$

* Many bridges are lacking in this respect. While the author would prefer to design bolts to resist tractive forces, some engineers do not consider it necessary.

The former may be neglected in this instance. Required area of bolts is,

$$40{,}000/11{,}000 \cdot 4 = 0.91 \text{ sq.in.; use } 1\tfrac{1}{4}'' \text{ dia. bolts.}$$

Detailing

Locate these bolts on center line of bearing as shown. Place first line of struts as near this as will allow drilling of holes for anchor bolts. Divide distance between gage lines of end

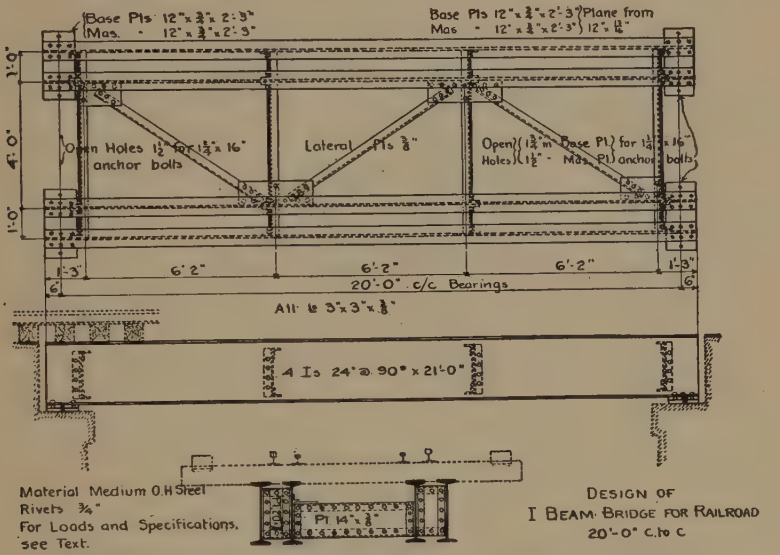


Fig. 71*n*.—Design of an I-Beam Span for a Single Track Railroad.

struts into three equal parts, if necessary altering position of end struts slightly to make divisions come in even quarter inches. This is to make laterals alike. (Art. 40.)

The structure may be shipped complete, Art. 48; to avoid field riveting, this will be done. The laterals should not be fastened to top of I-beams on account of interference with ties. To connect with bottom of top flange would mean bent plates which are undesirable, Art. 42. We shall therefore drop them down about 8 inches as shown. It will be noted that some of the rivets in the lateral plates cannot be driven

after assembly. We will therefore assemble and rivet up central struts and laterals; then rivet beams to this.

Two channels 12″ @25# ×1′-8″ and 1 channel 15″@ 33# ×3′-10¾″ might be substituted for the built-up struts. They would be slightly more economical, without materially changing strength.

We have treated this simple problem with some detail in order that the student might learn the practical application of the principles which he has just studied.

Remarks about References

For the purpose of arranging in order, the author has often scaled or estimated some dimensions. Therefore, too great a reliance must not be placed upon them. The idea is to help the *designer to select those references which he will need;* thus plate girders of each type will be given in order of span length.* These quantities, type and span length, are usually pre-determined, and the engineer is thus enabled to find what has been done before in a similar case. It is to be regretted that many times these published designs are oddities rather than standard practice. Also, references more than fifteen years old must be regarded by inexperienced men with caution; nevertheless, they may contain valuable data and will be often mentioned. In general our aim will be to give a *few good references on each subject.*

REFERENCES FOR RAILROAD I-BEAM BRIDGES

Engineering News, Vol. 35, p. 299; General.
Engineering News, Vol. 49, p. 482; Standard Plans.
Railroad Gazette, Vol. 38, p. 248; Standard Plans.
For open floors see references for Art. 76.

Prob. 71. Use a single Bethlehem I-beam on each side in above example and compare weight and shop cost.

* Arts. 90 and 93.

Art. 72. I-Beam Bridges with Solid Floors *

The essential idea of a solid floor bridge is to provide a support so that track may be laid and maintained as elsewhere. Gravel and broken stone are common materials for the ballast and may be estimated at 100 lbs. per cu.ft. Allow for any possible increase of depth or other additional weight, for example, that due to the presence of water. Great pains should be taken to prevent the latter by leaving open holes where the drip will cause no damage.

Unprotected steel carrying ballast, whether drained or not, *rusts rapidly and cannot be readily inspected or painted.* The brine dripping from refrigerator cars is especially destructive. A suitable protection is afforded by 2 inches of asphalt concrete sloped to drain holes or by an equal amount of ordinary cement concrete. The author believes that it is often practical to keep all steel encased or to employ reinforced concrete. However, construction of this kind is not within the scope of this work.

The arrangement of ties and ballast is like that on solid ground. No guard rails or timber need be used. Not less than 6 to 8 inches of ballast should be underneath ties. The effect of this track is to distribute load and the floor system may be computed on the assumption that the maximum wheel load is distributed over a space 3 feet long and 18 inches on either side of the rail. Impact allowance should be reduced or eliminated altogether. It may be wise to figure bridge for any possible location of rails. Whether it is or not, will depend on structure, arrangement of tracks in vicinity, and supervision by engineering department.

These solid floors constitute in a way horizontal plate girders, which amply suffice for lateral bracing, However, end struts are usually necessary.

We shall divide the various kinds of solid floors into four classes.

(1) *Trough.*
(2) *Flat Plate.*
(3) *Buckled Plate.*
(4) *Reinforced Concrete.*

* Journal Western Society Engineers, Vol. 10, p. 227.

Figs. 72*a*, 72*b* and 72*c* well represent the many varieties of Type (1). Ballast, tie, and rail are also indicated. The second may be placed entirely above troughs, or, if clearance is important, in the depressions as shown. In latter case, care must be taken to provide enough clearance for tamping ties. The usual thickness of steel is $\frac{3}{8}''$ to $\frac{1}{2}''$. However, sizes and rivets should be computed as carefully as in plate girders; weight in length l must be carried by beam of transverse width l. Approximate or exact formula may be used, the vertical plates constituting the webs, the angles and horizontal plates the flanges.

In through bridges, connection is made to girders by means of angles shop riveted to that part of the section which is concave upwards, thus allowing flooring to serve as a platform

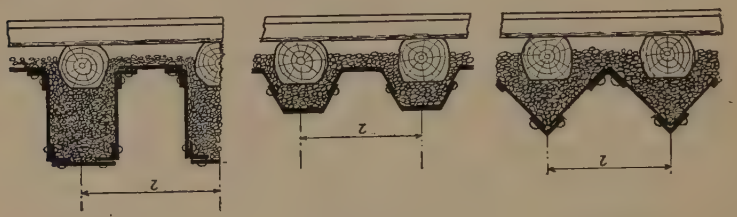


FIG. 72*a*. FIG. 72*b*. FIG. 72*c*.

Forms of Trough Solid Floors.

for driving field rivets. Type shown in Fig. 72*b* is sometimes carried on a continuous shelf angle. In deck bridges the section should be stiffened at points of support by angles and a diafram be placed across the ends for the double purpose of bracing section and keeping ballast in, Fig. 82*a*. Sections are usually shop riveted to form shipping pieces 6 to 8 feet in width which are riveted together in the field. By designing splice as seen in Fig. 72*d*,* notable economy may be effected in field riveting. No platforms are needed for the man who holds the rivet. There is no danger to traffic underneath. The heated rivets may be much more easily passed, and the whole gang will be together on a solid footing. For given loads, the cost is practically unchanged.

* Designed by A. L. Lee, C.E.

EXAMPLE

Let us compute a solid floor like Fig. 72*a* with a span of 16 feet when loaded with Cooper's E 60, for allowable stresses of 10,000, 7500 and 15,000 lbs. per sq. in. in flexure, shear, and bearing respectively. For dead load allow the following amounts,

FIG. 72*d*.—Economical Method of Splicing Trough Sections.

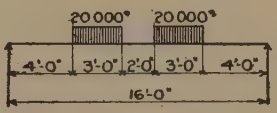


FIG. 72*e*.—Distribution of Live Load.

Track	60	lbs. per sq.ft.
Steel	75	" "
15 ins. ballast	125	" "
Total	260	" "

We will make distance l 2 feet.

Dead moment for strip $l = 520 \cdot 16 \cdot 16/8 = 16{,}600$ ft.lbs.
Live moment for strip $l = 20{,}000\ (7 - 1.5) = 110{,}000$ "

126,600 "

See Fig. 72*e*.

From Art. 71, $h = 1.75\sqrt{M/S} = 1.75\sqrt{\frac{126{,}600 \cdot 12}{10{,}000}} = 22''$.

$a = 126{,}600 \cdot 12/22 \cdot 10{,}000 = 6.90$ sq.in.

Use 2*Ls*, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, gross area 4.96, net 4.21 sq.in.
Use 1 Pl. $12'' \times \frac{3}{8}''$, gross area 4.50, net 3.75 "

Totals, gross area 9.46, net 7.96 "

But as there is excess metal and as it is too thin to resist corrosion and stand weight, we will decrease depth, making it 16″.

$$a = 126{,}600 \cdot 12/16 \cdot 10{,}000 = 9.50 \text{ sq.in.}$$

Use 2 *Ls*, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$,	gross area	5.74, net 4.87 sq.in.
Use 1 Pl. $12'' \times \frac{1}{2}''$	gross area	6.00, net 5.00 "
Totals,	gross area	11.74, net 9.87 "

The maximum shear is 24,200 lbs; we will use a $16'' \times \frac{3}{8}''$ web which gives a unit stress of 4000 lbs. per sq.in. on gross area.

Two rows of rivets of $\frac{7}{8}''$ dia. in single shear will carry the horizontal shear. The value for each of these rivets is 4500 lbs.

Horizontal shear per lin.in. is $24{,}200/16 = 1500$ lbs.

Considering one-half at top and one-half at bottom, maximum vertical force per lin.in. is:

$260/12 + 10{,}000/36 = 300$ lbs.
Resultant 1530 lbs.
Spacing must not exceed $9000/1530 = 5.87$ in.

When flooring rests upon rather than frames into its supports, stiffeners should be employed; they are seldom necessary elsewhere.

Type (*2*) *Flat Plate*

Here there is but one method. In this the flat plate is riveted onto I-beams which are spaced such a distance apart that the plate considered as a simple beam between flanges is not overstrained. Let us design a floor of this kind for the data of the last problem. Tabulate as follows. Note that third column must be obtained by estimating, trying, and revising if not correct. Moments and shears per lin. ft. of bridge will be unchanged. Maximum load per sq.in. on plate is 25 lbs.

Thickness. Plate.	Allowable Span.	Try I-Beam.	Distance c to c.	Flexural Unit Stresses I-Beam.	Shearing Unit Stresses. I-Beam.	Weight per Sq.Ft.	Remarks.
$\frac{3}{8}''$	9″	20″ @ 65#	15.25″	8250	1540	67#	
$\frac{7}{16}$	10.5	20 @ 65	16.75	9000	1700	64	
$\frac{1}{2}$	12.0	20 @ 65	18.25	9870	1850	63	Best
$\frac{9}{16}$	13.5	20 @ 70	19.83	10280	1720	65	

It is doubtful if the last one ought to be allowed to stand on account of high flexural stress in I-beams. It will be noted, however, that we have not considered the plate as assisting the I-beam. Really it does help a great deal but practice is to ignore this.

Choice does not necessarily fall on one with the least weight. The larger spans will have fewer parts and rivets and this will tend to decrease shop work and consequently cost per pound.

When flooring is supported by a through structure, the I-beams connect thereto by the usual two angles shop riveted to I's. For a deck structure, I-beams should be braced together by channel struts, and angle placed on the plate to retain the ballast, see Fig. 82*b*. Plates are usually arranged to make one length stretch between supports. The widths are chosen to allow an I-beam to serve as a splice, nothing further being necessary. Plates and I-beams may be field riveted, or else they may be shipped in sections of two or three I-beams, shop riveted to plates.

Type (3) *Buckled Plate Floors*

Here the span may be much increased due to the extra strength imparted by the buckles. In other respects, it is much like the flat plate floor.

Let us suppose a square buckled plate with the buckles down to be subjected to a uniform load. Let a section of the dome along either center line be a parabola, and let Fig. 72*f* represent a section of it one unit long in a plane perpendicular to the paper. From the properties of the parabola and similar triangles,

$$H : V = \tfrac{1}{2}l : 2r \quad \text{or} \quad H = Vl/4r.$$

But $V=\frac{1}{2}wl$ where w equals load per square unit. Also, since r is usually small, $H=St$ where S is the unit stress and t the thickness. Substituting,

$$St=wl^2/8r \quad \text{or} \quad w=8Str/l^2.$$

Now *assuming* that each linear unit of the periphery carries the same stress, we have that the total weight W,

$$=\frac{8Str}{l^2}\times l\times 2l=16Str, \quad \text{buckles } down.$$

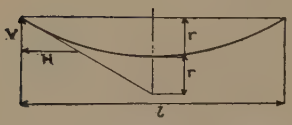


FIG. 72f.—Section of a Buckled Plate.

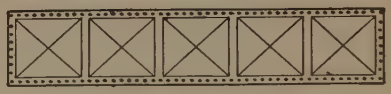


FIG. 72g.—A Buckled Plate.

If the buckles are turned up, the safe load is lessened on account of the lower allowable stresses for thin plates under compression,

$$W=4Str, \quad \text{buckles } up.$$

If fastened on two sides instead of four, halve above values. If supported but not fastened, halve results previously given.

Buckled plates are $\frac{1}{4}$ to $\frac{1}{2}''$ thick. $\frac{3}{8}''$ is common for railroads. Buckles are from 2 to 5.5 feet square and have 2 to 3.5 ins. rise. They usually come in a long plate, Fig. 72g, one buckle plus two margins in width and n buckles with $n+1$ margins in length. n may be any number from 1 to 15. Tables in the Carnegie handbook show available sizes. Plate should be fastened to stringers with rivets not less than $\frac{5}{8}''$ dia. If it is desired to obtain the strength of a plate fastened on all sides, an angle or T should be riveted on at each end and intermediate margin. The effect of the usual arrangement

of floors is such that it distributes the loads enough so we need not consider concentrations on one part only of a buckle.

Designing a floor of this type for the same problem as heretofore, we find that 24″ I's @95 ℔, spaced 2′ 6″ center to center will carry the load. Taking one wheel as concentrated on a single panel 30″ square with a rise of 2″ and a thickness of $\frac{3}{8}$″, we get a unit stress of,

$$S=\frac{W}{16tr}=\frac{30{,}000}{16\cdot\frac{3}{8}\cdot 2}=2500 \text{ lbs. per sq.in.}$$

As this will probably be supported on two sides only, allowable

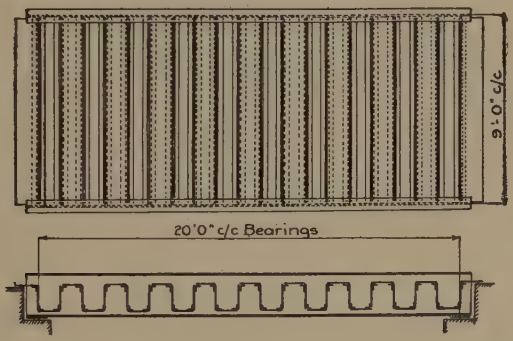


Fig. 72*h*.—Type (*1*). A Trough Section Solid Floor for a Single Track Railroad.

stress is 10,000/2 = 5000, O. K. Estimated weight per sq.ft. is 53 ℔.

Type (*4*) *Reinforced Concrete Floors*

While the author has a high opinion of these floors, he believes their consideration here to be inappropriate. An arrangement which protects the beams is desirable. There can be no objection to this design which should make the whole bridge very durable. There are two methods,

(*4a*) Corresponds to Type (*2*) with slab of reinforced concrete supported by I-beams.

(*4b*) Runs from girder to girder.

At abutments, the I-beams and trough sections are provided with bearings similar to those for I-beam spans. Joint between bridge and masonry is arranged as shown in Fig. 72*h*.

In the above we have considered solid floors in general. We will now show in Figs. 72*h*, 72*i*, and 72*j*, the adaptation of solid floor to railroad I-beam spans.

In Fig. 72*j*, dotted lines show proposed addition to render structure durable.

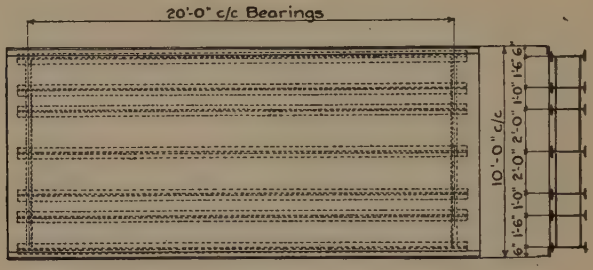


FIG. 72*i*.

Type (*2*) An I-Beam and Plate Solid Floor for a Single Track Railroad.

FIG. 72*j*.

Type (*4a*) A Reinforced Concrete Floor for a Single Track Railroad.

REFERENCES

Solid Floors for Railroads in General

American Society of Civil Engineers; Vol. 27, p. 483.

Engineering News; Vol. 28, p. 386; Vol. 32, pp. 399, 401; Vol. 50, p. 437; Vol. 53, pp. 161, 162.

Journal of the Association of Engineering Societies; Vol. 15, p. 232.

Journal of the Western Society of Civil Engineers; Vol. 10, p. 227.

Railroad Gazette, Vol. 39. p. 104.

Journal Cleveland Engineering Society, November 1912.

Examples of I-Beam Bridges with Solid Floors

Engineering News; Vol. 63, p. 158.
Engineering Record; Vol. 61, p. 101; Vol. 62, p. 310.

Prob. 72. Design solid floor of Fig. 72*h* for the same loads and stresses as example in text.

Art. 73. I-Beam Bridges for Highways—Plank Flooring

There are two principal types of highway bridges:
(1) *Plank Floor.*
(2) *Paved Floor.*

Both are supported in the same manner—by I-beams parallel to the roadway with simple bearings of masonry and sole plates, fixed at one end and free to expand at the other.

Fig. 73*a*.—Sketch Showing Distribution of Stress in Tension Flange of an I-Beam and Plate Section.

Channels are sometimes used at the side because with the same depth they have about half the strength which corresponds in a way to their loading. I-beams with plates riveted on them may also be employed; if this is done, two holes should be deducted from flanges, even though rivets be staggered at section of maximum moment. We are not justified in assuming that flange can transfer stress from one side to the other without computing moment caused thereby. If we allow for this, it is practically the same as deducting both holes, see Fig. 73*a*. We believe in general that it is not an efficient beam.

Plank Floor

Here the traffic runs directly on the plank. There may be two layers, a wearing surface of $\frac{7}{8}$ to $1\frac{3}{4}''$ stuff, to be replaced as fast as worn out, and a lower layer computed to carry the load. This is the better design for roadways. However,

one layer is often employed. It is then computed to carry the load and must be replaced long before it is entirely worn out. Thickness should never be less than $2\frac{3}{4}''$ or $\frac{1}{15}$ the span for roadways, or $1\frac{3}{4}''$ and $\frac{1}{30}$ for sidewalks. Yellow pine and oak are the best woods. The lower layer may be expected to last from 6 to 12 years, while upper layer will have to be replaced every six months to five years according to the amount of traffic. The plank may be fastened to I-beams by clinching

FIGS. 73*b*. 73*c*. 73*d*. 73*e*.

Methods of Fastening Plank to Stringers.

nails around the steel or by either of the three methods shown in Figs. 73*b*, 73*c*, or 73*d*. The first, while cheaper, is not as good as the others. Obviously, any of these methods might also be applied to channels. In Fig. 73*b*, lag screws driven upwards may be substituted to advantage for the bolts. Clips should be opposite in Fig. 73*c*. Type shown in Fig. 73*d* is poor for I-beams but much better for channels, Fig. 73*e*.

Single plank or lower surface is laid transversely to stringers, while wearing surface is arranged diagonally or longitudinally.

FIG. 73*f*.—Arrangement of Floor for Drainage.

Openings of $\frac{1}{4}$ to $\frac{1}{2}''$ should be left between plank to allow drainage, thus preventing wet rot. (Art. 3.) The spacing of the bolts should not exceed 4 feet.

A slope to the floor will be necessary for drainage. Fig. 73*f* shows a desirable outline. There are three ways of providing variation in height:

(*1*) By altering elevations of masonry support.

(*2*) By use of "chairs" (cast bearings) on flat plates at abutments.

(*3*) By variation in nailing pieces.

As either method involves trouble and expense, one or both sidewalks are omitted and frequently floor is made flat. This allows the accumulation of water which renders travel unpleasant and hastens decay of wood.

Fig. 73*g* shows arrangement at abutment. Room must be provided at *a* for expansion and variations in fabrication of beams and in building masonry.

FIG. 73*g*.—Arrangement at Abutment.

The plank should be computed by considering as a simple beam subjected to the maximum load, distributed over 24″ in breadth. If there are two layers, twice the shearing strength of 12″ in breadth of the upper layer may be added to its capacity. For example, let it be required to find the stress caused by a concentrated load of 6000# on $2\frac{3}{4}$″ yellow pine plank with $1\frac{3}{4}$″ sheathing of the same material when supporting beams are spaced 3 feet center to center.

Capacity of top layer in shear $= \frac{2}{3} \cdot 2 \cdot 1\frac{3}{4} \cdot 12 \cdot 70 = 1960$ lbs.

$$6000 - 1960 = 4040 \text{ say } 4000 \text{ lbs.}$$

Maximum flexural stress

$$= \frac{6M}{bd^2} = \frac{6Pl}{4bd^2} = \frac{6 \cdot 4000 \cdot 36}{4 \cdot 24 \cdot 2.75 \cdot 2.75} = 1200 \text{ lbs. per sq.in.}$$

Maximum shearing unit stress

$$= \frac{3P}{2bd} = \frac{3 \cdot 4000}{2 \cdot 24 \cdot 2.75} = 91 \text{ lbs. per sq.in.}$$

and we find values higher than those stated for bridges in Art. 6, Vol. 1.

A very common rule for plank is to give it a thickness of one-twelfth the span. To show method of determining economic spacing, let us consider an I-beam span of 20 feet, using above

rule. Uniform load per sq.ft. including dead weight, 200#. Allowable flexural stress for beams, 12,000 lbs. per sq.in. Costs, 5 cents per ft., B.M. for timber; 3 cents per lb. for steel. Letting d represent spacing in feet.

Cost of timber per sq.ft. in cents $= 5d$.

Cost of steel per sq.ft. in cents $= \frac{21}{d}\sqrt{\frac{200 \cdot 20 \cdot d \cdot 240}{8 \cdot 12{,}000}} = 66.5/\sqrt{d}$

(Derived from $w = 7\sqrt{M/S}$, Art. 71).

Total cost per sq.ft. $= 5d + 66.5d^{-\frac{1}{2}}$.

Differentiating for a minimum we find that d equals 3.5 feet. The customary spacing is 2 to 3 feet. It would naturally be less since above discussion ignores increased maintenance cost of timber flooring.

In locating stringers, particular attention should be given to supporting rails of an actual or proposed electric car line. Ties carried by I-beams somewhat farther apart than the rails may be used, Fig. 73*h*, as in railroad open floors, or rails may be

FIG. 73*h*.—Floor for a Highway Bridge.

fastened to beam stringers directly underneath with or without the intervention of wood.

Sway bracing, Fig. 71*j*, of channels riveted in between stringers must be inserted at the ends; it would be better if intermediate bracing were located at intervals of not more than 15 feet. X bracing, Fig. 71*i*, may also be used instead. For such small frames, the additional shop cost more than balances saving in material; hence first named is better. If care is taken in fastening planking, no lateral bracing will be necessary. In this case, it is better if top surface is laid diagonally.

REFERENCES

"Design of Plate Girders," International Library of Technology.

Prob. 73. Determine depth of lower layer of yellow pine for a span of 4 feet and a concentrated load of 10,000 lbs. Upper layer is $1\frac{3}{8}''$ white oak.

Art. 74. I-Beam Bridges for Highways—Solid Floors

Plank floors wear very rapidly under heavy traffic. On the other hand, they are light and comparatively noiseless. Stone, wood, brick, and asphalt are the common materials used for paving. We shall not discuss the relative merits of these different types further than to roughly indicate weights. These are in lbs. per sq.ft. for an inch in thickness.

Timber	4	Asphalt	8
Sand	8	Brick	12
Concrete	12	Granite	14

Weight and thickness of pavements, not including the steel, will vary with the details. Following may be taken as very rough average values.

	Roadway.				Sidewalk.			
	Thickness in Inches.			Weight in Lbs. per Sq.Ft.	Thickness in Inches.			Weight in Lbs. per Sq.Ft.
	Concrete.	Sand.	Surface.		Concrete.	Sand.	Surface.	
Asphalt	4	..	2	64	3	..	$1\frac{1}{2}$	48
Asphalt block	4	1	3	80	3	1	3	68
Wooden "	4	1	4	72	3	1	3	56*
Brick	4	1	4	104	3	1	3	80
Stone	4	1	6	140	Un	desira	ble	

* Wooden block may also rest on plank.

Although the three other types of solid floor, Art. 72, are

employed to some extent, a *buckled plate* floor is the usual design. The depth of concrete is determined by minimum allowable thickness above the steel not including rivet heads. This distance should be at least 3 inches for roadway and 2 inches for sidewalk. Thickness of buckled plate should be not less than $\frac{5}{16}''$ and $\frac{1}{4}''$ respectively. As these give plenty of strength, they are very common. A sixteenth extra may be employed where a high grade job is desired.

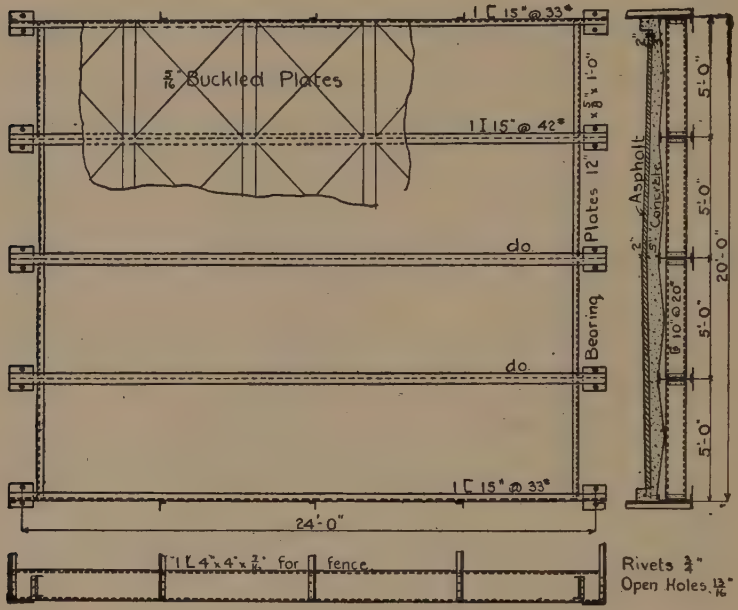


FIG. 74.—Typical I-Beam Bridge for a Highway.

NOTE.—Holes should be left in center of buckled plate at intervals, so that rivets may be heated on top of floor and dropped through to "bucker up" underneath.

Three to five feet apart is the usual spacing of the stringers. Provision should be made for electric cars by inserting a stringer under each rail. Latter may be fastened by bolts or clips to the stringer direct.

The varying elevation of the surface which is necessary as in plank bridges, may be obtained by increasing thickness of

concrete, by varying heights of masonry, or by the use of chairs. One of the last two methods seems preferable.

REFERENCES

Solid Floors for Highways in General

Engineering News; Vol. 55, p. 266.
Engineering Record; Vol. 66, p. 647; Vol. 68, p. 640.
Journal of the Association of Engineering Societies; Vol. 15, p. 232.
Engineering and Contracting; Vol. 39, p. 221; Vol. 40, p. 43.

Examples of I-Beam Bridges with Solid Floors

Engineering News; Vol. 41, p. 50; Vol. 44, p. 412; Vol. 52, p. 581.
Engineering Record; Vol. 56, p. 565; Vol. 64, p. 67; Vol. 66, p. 606.

Prob. 74. A beam bridge is designed with a plank floor for live load of 80 lbs. per sq.ft. and a dead of 20. What will be the percentage increase of weight in the beams if solid floor weighing 100 lbs. per sq.ft. be substituted for the plank?

Art. 75. Fences

Obviously, a fence will always be necessary for a highway bridge in order to prevent teams and pedestrians from accidentally falling from the bridge. Sometimes a plate girder projects enough to serve as a barrier, hardly an ornamental one, it is true. Occasionally a truss is so placed that it will answer the purpose; seldom, however, will it do more than furnish the posts.

The following are the desirable qualities for a fence:

(*A*) It should have no openings in the lower portion such that a small child could crawl through. Say nothing over 6″ in both directions.

(*B*) It should be able to withstand the push of a crowd of people or the wind. Obviously we would not expect both to

occur simultaneously. The former may be taken at 60 to 120 lbs. per lin.ft., varying with the locality, applied horizontally at the top of the rail and acting from the inside out. We will estimate the latter as 40 lbs. per sq.ft. of exposed surface and allow it to act in either direction.

(*C*) It should be economical.

(*D*) It should be æsthetic.

Taking up the latter, first as applied to all structures, its essentials are:

(*a*) *Harmony*, that is, agreement with surroundings. Thus a bridge in a rustic park might well be made of quarry faced stonework or of boulder faced concrete. The same agreement should exist between different parts of the bridge.

(*b*) *Sincerity*. It is now considered best to show *frankly* the source of strength. Violations of this rule are very frequent when a pleasing design is attempted. As an instance of such, we will cite the construction of an apparent stone arch around a plate girder to prevent the unsightly appearance of the latter.

(*c*) *Symmetry*. This is desirable but of course economic considerations often absolutely forbid. For example, a fence should have a certain number of panels for a panel length of the bridge.

(*d*) *Grace*. Choose lines which are not too rigid. Curved are better than straight lines.

(*e*) *Conception*. We shall use this rather general term to represent that artistic idea of which the structure is the expression. For example, the Soldiers Memorial Building in Pittsburgh conveys the impression of a majestic monument.

(*f*) *Ornamentation*. This should be the complement of the preceding, that is, it should fill out the conception in the details. Avoid an excess, using it simply to relieve the monotony of bare walls and too rigid lines.

In general, appearance receives too little attention. Of course safety must come first and the call for economy is usually present. However, if but a small sum is needed to satisfy æsthetic requirements, the fact should be called to the client's attention. The locality makes a great deal of difference.

Taking up the subject of fences once more, we will find few that will conform to these requirements. In fact there are some localities where they may be partially disregarded. The railings for an isolated country bridge need not provide absolute safety for an unprotected child. A structure for workmen only need have but a single rail. Not much strength will be required, as employees are expected to possess enough intelligence not to overstress it.

We may divide bridge fences into four classes:

(1) *Wooden Fences.*
(2) *Pipe Fences.*
(3) *Fences of Structural Steel.*
(4) *Fences of Ornamental Steel.*

The usual height of the top of fence is 3′ 6″ above the floor. Spacing of posts will vary a great deal. It should not exceed eight feet for the first two classes. Ten feet may be taken as an average in the last two. Below we give typical examples of each class.

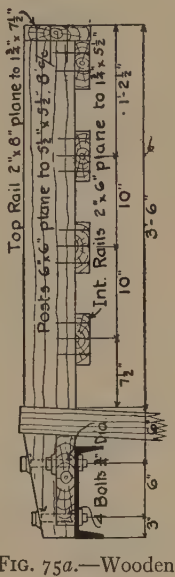


FIG. 75*a*.—Wooden Fence.

Type (1) *Wooden Fence*

This fence, Fig. 75*a*, consists of posts about square, to which are nailed the intermediate rails, planks about 2″×6″. The top rail may be made of two plank, one horizontal, the other vertical, nailed together. The posts are fastened at the base by bolting to the outside stringer. (Commonly called the *fascia* girder.)

Let us compute stresses caused in design shown, Fig. 75*a*, by a wind pressure of 40 lbs. per sq.ft. and 100 lbs. per lin.ft., applied horizontally at the top.

In intermediate rails, considering a strip 1 inch wide,

$$S=\frac{6M}{bh^2}=\frac{6\cdot 8\cdot 40\cdot 96}{8\cdot 12\cdot 1\frac{3}{4}\cdot 1\frac{3}{4}}=630 \text{ lbs. per sq.in.}$$

Assume horizontal joist in top rail to carry entire horizontal load,

$$S=\frac{6M}{bh^2}=\frac{6\cdot 8\cdot 100\cdot 96}{8\cdot 1\frac{3}{4}\cdot 7\frac{1}{2}\cdot 7\frac{1}{2}}=590 \text{ lbs. per sq.in.}$$

For posts,

$$S=\frac{6M}{bh^2}=\frac{6\cdot 800\cdot 48}{5\cdot 5\cdot 5\cdot 5\cdot 5}=1380 \text{ lbs. per sq.in.}$$

This is not excessive if yellow pine be used. We have not made deductions for two $\frac{3}{4}''$ bolt holes. On the other hand, the maximum loads are rather infrequent and post will be aided as described below.

For bolts fastening posts to fascia girder, stress in upper pair equals

$$\frac{800\cdot 54}{6}=7200 \text{ lbs.}$$

Use 2 bolts, $\frac{3}{4}''$ dia., top and bottom and toenail plank to post. Note that the net section of bolts is at screw thread.

Type (*2*) *Pipe Fences*

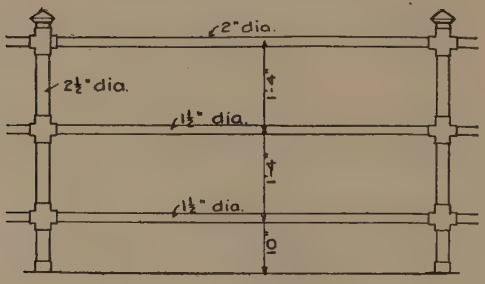


FIG. 75*b*.—A Pipe Fence.

These consist of rails and posts of ordinary pipe, screwed into or through double tees. An ornamental casting is placed on top, while one which admits of fastening to plank is at the bottom. Horizontal rails should be continuous.

It is somewhat difficult to differentiate between Types (3) and (4). We shall classify as ornamental those fences which contain bent work. In either one, rivets and material under the usual minimum sizes and thickness are quite frequent. We find two kinds of posts: the *beam* post attached to the fascia girder; and the *braced* post, Fig. 75*c*, fastened to the sidewalk floor beam. The inclined member should be an angle; a rod, while frequently used, will seldom possess the

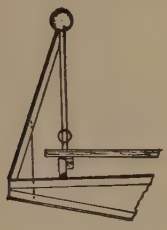


FIG. 75*c*.—Braced Post.

FIG. 75*d*.—Hand Rail.

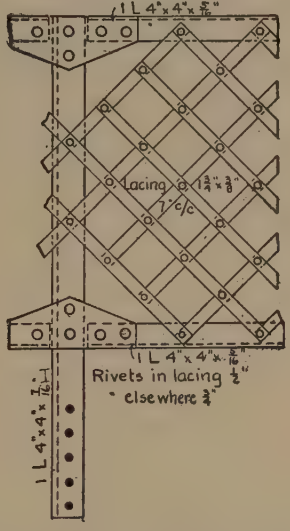


FIG. 75*e*.—Fence of Structural Steel.

necessary strength to resist compression. Some provision for adjustment is desirable that fence may be kept in exact alignment.

Type (3) *Fences of Structural Steel*

We may again subdivide. First we have fences much like Type 2 except that angles may be employed for intermediate and lower rails, and a hand rail, Fig. 75*d*, an ordinary T, one angle, or two angles for top rail. Secondly, top and bottom may be either a T, two angles, or a single angle, with latticing in between them composed of flats or small angles, Fig. 75*e*.

FIG. 75*f*.

Ornamental Fence.

Smithfield St. Bridge over the Monongahela River, Pittsburgh, Pa.
Wabash Cantilever Bridge in the distance.

FIG. 75*g*.

Ornamental Fence.

Shady Ave. Plate Girder Bridge over the Pennsylvania R. R., Pittsburgh, Pa.

Type (*4*) *Fences of Ornamental Steel*

We shall not attempt to classify the many types of ornamental fences. Figs. 75*f* and 75*g* give a general idea. The usual operations are bending, twisting, and riveting.

Because of small material and large amounts of bending and so forth, costs per pound for structural fences runs rather high, varying from 4 to 8 cents per pound. The expense per lineal foot for the different types will vary a great deal with the design. It may be taken quite roughly as:

Type (*1*)	$0.50	Type (*3*)	$1.50
Type (*2*)	1.00	Type (*4*)	2.50

REFERENCES

Engineering News: Vol. 31, p. 224; Vol. 42, pp. 92, 226; Vol. 55, p. 266.

Engineering Record; Vol. 29, p. 267.

Prob. 75. Design a steel angle fence with loads and principal dimensions same as for Fig. 75*a*. Allowable stresses as in Art. 92.

Art. 76. Open Floor Plate Girder Bridges for Railroads

From the upper limit for I-beams up to the point where riveted trusses become more economical, plate girders are the accepted design.

Plate girders are used in bridges, foundations, buildings, stand-pipes, cranes, and other places. We shall consider the first case only in this chapter. Again, they may be employed for beam bridges, movable bridges, arches, or as parts of a bridge of some kind. However, the term " Plate Girder Bridge " as ordinarily used, signifies a simple beam bridge where the main carrying pieces are plate girders. When supported by columns, they are termed " Viaducts," and form the subject of the next

chapter. The principles developed herein are generally applicable to all, perhaps with some slight modification.

Plate girder bridges may be subdivided according to use into *railroad* and *highway;* according to floor, into *open*, *plank*, and *solid;* according to position of traffic, into *deck*, *half through*, and *through*;* these terms have already been explained in describing I-beam bridges.

Through Bridges

The half through bridge will be taken up in detail in the following articles. The through bridge resembles it as far as the girders and their shoes are concerned. The floor system is omitted altogether, the ties being given sufficient depth to carry the loads to the bottom flanges of the plate girders on which they rest.

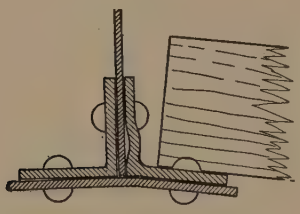


Fig. 76*a*.—Effect of Resting Tie on Flange of Plate Girder.

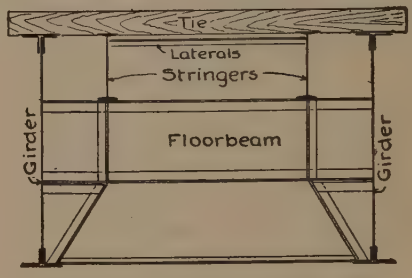


Fig. 76*b*.—Unusual Type of Deck Plate Girder.

This method is not in favor except for solid floor bridges, Art. 82. The objections are:

(*1*) Bending stresses in the flange angle, Fig. 76*a*.

(*2*) Tensile stresses in rivet.

(*3*) Warping of timber.

(*4*) Danger of overturning deep ties in case of derailment.

An advantage which by no means offsets its faults is the small distance between base of rail and under side of bridge.

* Through plate girders with ties resting on bottom flange are seldom built nowadays, and the term " through " is so generally used for what we have termed the " half through," as to give it the sanction of common usage. However, we will retain the above meanings.

The bottom lateral system is made of angles figured to carry tension only, as in half through bridges. Since there are no floor beams to act as compression members, struts must be designed for this purpose alone. They must be made as deep as possible, since, in addition to the wind loads, they stiffen the top flange by means of gussets, Fig. 70*c*. The force of the latter may be computed by using proof of Art. 56. But in this case the gussets have the additional duty of holding the entire girder vertical. Due to inaccuracies of workmanship, it may not be exactly plumb, and the load may not be applied in exactly the computed direction. Hence we shall estimate the lateral force to be increased from $\frac{1}{30}$ to $\frac{1}{10}$. Therefore

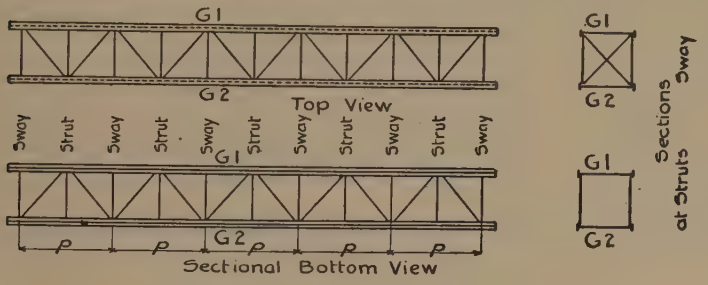


Fig. 76*c*.—Bracing of Deck Plate Girders.

one-tenth safe compression in top flange as a short strut is considered to be applied horizontally to top chord. In a varying flange, we regard the uniform load per lineal foot applied horizontally at any point as the safe compressive strength of the flange at that point, considered as a short strut, divided by ten times the entire span in feet. The gussets may be computed as beams of variable depth of which the stiffener angles form one chord and the gusset angles the other.

Deck Bridges

These have been built with stringers and floorbeams, the former on a level with the girders, Fig. 76*b*. The usual type without either of these is shown in Fig. 70*b*. Plate girders and shoes for the same are similar to those for half through

spans to be taken up in the following articles. The bracing is different, however, the three complete systems being used.

6′ 6″ is considered the best spacing for supporting ties and this is the usual distance center to center of girders in spans less than 80 feet. For greater lengths, due in part to increased depth, there is a lack of stiffness and stability unless spacing is made one-twelfth span or more.

Taking up now the three systems of bracing, the *upper lateral*, the *lower lateral*, and the *sway*, we may note that they form an indeterminate system. This is usually solved in practice by ignoring sway bracing except at ends. It is customary, however, to insert all systems except in short plate girder bridges, where lower laterals may be omitted. While bracing is usually figured for wind and centrifugal force alone, note

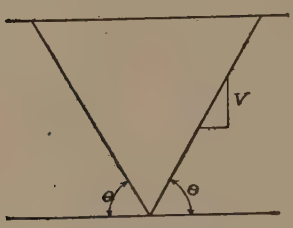


FIG. 76*d*.—Economy of Bracing.

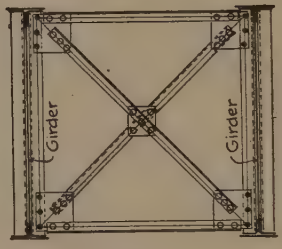


FIG. 76*e*.—Typical Cross Frame.

that it also has to brace top chord as already considered for through bridges.

Fig. 76*c* shows a typical arrangement. The most economical angle for any bracing is about 45 degrees. To prove this, let it be required to carry a given shear, V, by a member inclined at an angle, θ, to the horizontal, Fig. 76*d*, and let S be the allowable unit stress. Amount of material per lineal unit, measured horizontally, is then,

$$\frac{V \csc \theta \sec \theta}{S} = \frac{2V}{S \sin 2\theta},$$

which becomes a minimum when θ equals 45 degrees. Stresses due to own weight and the reduction from the compression

formula tend towards increasing this angle while shop and field work are favored by decreasing it.

Hence we leave it about 45 degrees, the spacing of the girders being determined as already noted. The need of supporting top flange at intervals not exceeding 15 times width of cover plate must also be taken into consideration. As this is not necessary for bottom flange, struts may be omitted except at cross frames.

A single angle is commonly employed for each member of the bracing. If too small or if radius of gyration is not large enough, Art. 50, *d* 9, two angles riveted together may be employed. In the former case, minimum radius of gyration for an angle must be taken, that is, that about a diagonal

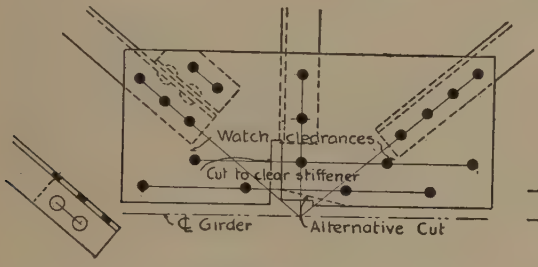


FIG. 76*f*.
Lateral Plate.

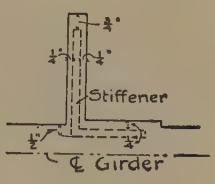


FIG. 76*g*.
Clearance for Cut in Lateral Plate.

axis. It is customary to use angles approximately equal legged. If one is longer than the other, place it vertically in order that more resistance may be offered to bending stresses due to own weight; however, this tends to make end connection eccentric. Upper laterals have vertical legs turned down to avoid interference with ties; lower have them turned up for appearance's sake. The longer legs of a two angle section should be placed together to increase the radius of gyration. Cross frames are commonly designed as shown in Fig. 76*e*, the diagonal angles being turned in opposite directions to make an easy joint at their intersection.

The erection of plate girder bridges has already been considered in Art. 49, Vol. I. Within the limit of shipping and erecting facilities they should be riveted up complete. This

is usually done for deck structures less than 60 or 75 feet. Above that limit, cross frames are shop riveted together and fastened in the field to lateral connection plates and to the stiffeners of the girders. Diagonals are field riveted at their ends. Lateral plates may be shop riveted to girder or shipped loose. Former cheapens erection but makes shipment more expensive and the plates are likely to be bent. Design structure so that the girders may be set in the proper place and connecting pieces "swung in," that is, revolved into position.

If there are more than four rivets, it is probably better to use a lug angle in the connection. See Art. 50, *d* 5. This is to lessen the eccentricity, some of which, nevertheless, still exists. It is seldom figured although present.

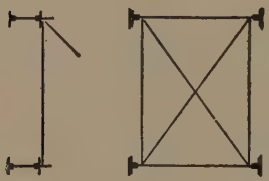


FIG. 76*h*.
Faulty Arrangement of Laterals at Bearing.
(*a*) Horizontal Section.
(*b*) Vertical Section.

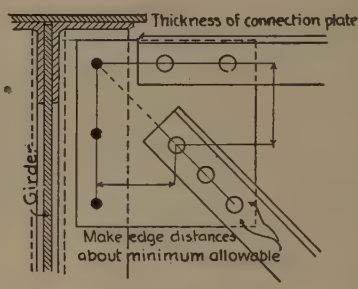


FIG. 76*i*.—Corner Plate in Sway Frame.

Fig. 76*f* shows a typical connection plate with the lug angle connection on the left. To avoid varying depths of cross frames and interference with ties, plate is placed inside of flange angles; it must hence be cut to clear stiffeners where they occur. This is a re-entrant cut, expensive in the shop, but we do not know how to avoid it. Allow clearances not less than those shown in Fig. 76*g*. So design cross frames that they may readily be swung in after the girders are set in final position, and be sure that the field rivets in lower end lateral plates may be driven. Sometimes the two objects are met by arranging bracing as shown in Fig. 76*h*, but a consideration of the stresses carried will show the fallacy of the design. A better way to meet the first requirement is to shorten

top and bottom struts and keep corner connection plates down, say $\frac{1}{4}''$, Fig. 76*i*. The distance back to back of angles in cross frames should be made $\frac{1}{8}''$ less than the figured distance between lateral plates and marked "*not more.*" To meet second requirement, we may rivet in cross frames before lowering girders onto shoe, very objectionable to be sure, but perhaps the best that can be done. A stronger and more compact joint at the expense of a little trouble may be obtained by running angles in closer to engage a rivet on the girder itself.

FIG. 76*j*.—Half Through Plate Girder Bridge of The Northern Pacific R. R. at Sappington, Montana. McClintic-Marshall Construction Co., Rankin, Pa.

A half through plate girder bridge may be divided into the following parts:

(1) *Stringers.*
(2) *Floor Beams.* } *Floor System.*
(3) *Web*
(4) *Flanges.* } *Plate Girders.*
(5) *Shoes.*
(6) *Bracing.*

Prob. 76. Design bracing for a span of 60 feet when arranged as seen in Fig. 76*c*. Allowable stresses in medium steel as given in Art. 50. Transverse horizontal load per lin.ft. 600 lbs. live on top chord, and 300 lbs. dead on bottom. Girders 8′ deep and 7′ c. to c.

REFERENCES FOR OPEN FLOORS

Engineering News, Vol. 38, p. 302; Vol. 52, p. 153; Vol. 62, pp. 270, 282.

Art. 77. Stringers for Open Floor Railroad Bridges

The floor beams of a half through plate girder divide the bridge into panel lengths, p, Fig. 77*a*, each in a way constituting

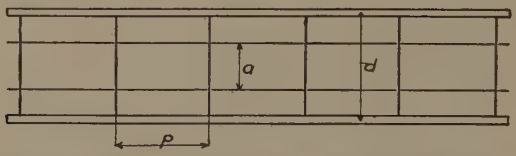


Fig. 77*a*.—Outline of Half Through Plate Girder Bridge.

an I-beam span of its own. d is made as small as possible with the necessary clearances. a is usually about 6′ 6″, and the track is laid on the stringers as explained in Art. 71. The distance p is determined by economical considerations.

Let dimensions be in inches and w be the load per rail in pounds per lin.in.; suppose further that stringers are rolled and floor beam built-up I's; then cost in cents per lin.ft. of floor system is, see Art. 71,

$$42\sqrt{\frac{wp^2}{8S}}+\frac{32d}{p}\sqrt{wp\frac{(d-a)}{2S}}.$$

Differentiating with regard to p for a minimum,

$$42\sqrt{\frac{w}{8S}}-\frac{16d}{p^{1.5}}\sqrt{w\frac{(d-a)}{2S}}=0.$$

Whence $p=0.83\sqrt[3]{d^2(d-a)}$ which holds for dimensions in any units.

For example, let $d=15'\ 0''$, $a=6'\ 6''$, then $p=10.3'$. For double track, we may likewise deduce where the four stringers are spaced a apart,

$$\text{Cost}=84\sqrt{\frac{wp^2}{8S}}+\frac{32d}{p}\sqrt{\frac{wp(d-2a)}{S}},$$

and economical $p=0.66\sqrt[3]{d^2(d-2a)}$.

If $d=28'\ 0''$ and $a=6'\ 6''$, then $p=15.2$ feet.

However, this does not take into account locomotive loadings, the increased cost in shop and erection of a large number of small parts, and economy in the main girders. All these tend to increase p. The usual amounts are 12 *to* 14 *feet for single* and 18 *to* 20 *for double track*. Always make panel lengths alike to decrease costs in drawing and templet room, Art. 40. Use, if possible, a span such that an I-beam of the minimum weight for given depth is fully stressed. Economy of material may likewise be promoted by limiting spans to such lengths that a single I suffices. There is also a saving in the main girder in the use of an odd number of panels since maximum moment is thereby reduced.

Where two I-beams are employed under each rail, they must not be placed too close together on account of the difficulty of driving the rivets which connect to the floor beam, Art. 49. They are hence arranged symmetrically about center line of the rail and at least $7''$ apart *in the clear*.

Built-up stringers are sometimes employed although we believe that they are seldom economical for this location. They are commonly made of a web and four unequal legged angles with longer legs horizontal. Cover plates will seldom be necessary, and as they mean considerable additional shop and track work, it is desirable to avoid them. Computations are similar to those for plate girders, Art. 54.

Bracing of Stringers

The main girders shield the stringers to such an extent that the guard timbers and rails can carry any wind load and render bracing needless for panel lengths less than 15 feet.

The customary connection to floor beam braces stringer at its ends. If the span exceeds 25 times width of flange, intermediate bracing must be inserted or allowable flexural stress reduced by compression formula, Art. 50, *d* 11. (Note that length has been increased to allow for support of ties.) Economical considerations will determine the proper course to pursue. Figure cost of bracing at a high price per pound, as it is small work, and will increase cost of shipping if shop riveted and that of erection if field riveted. It may be noted that top stringer laterals not only carry the wind but also serve to stiffen the top flange. Both objects may also be accomplished by attaching lower girder laterals to bottom of stringers and then using sway bracing between stringers at those points.

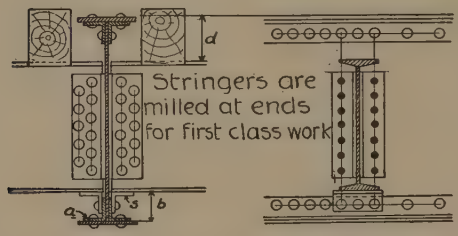


Fig. 77*b*.

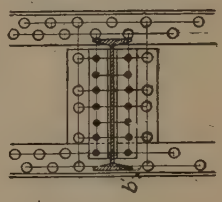


Fig. 77*c*. Fig. 77*d*.

Connection of Stringers to Floor Beams.

Connection to Floor Beams

Usually it will not interfere with economy if the floor beams are made enough deeper than stringer so that latter can frame directly into the web of the former, Fig. 77*b* and 77*c*. *d* should not be more than depth of dapped tie, while *b* should be such as to allow bottom laterals to pass underneath and also to give proper values to gages in angles *a* and *s* if a shelf angle be employed. The latter is preferable for ease of erection as explained in Art. 49, *g*. On account of the difficulty of obtaining workmanship accurate enough to ensure two sets of rivets acting in unison, it is better to ignore the strength in *s*.

An additional plate as shown in Fig. 77*d*, is sometimes economical because it may lessen the number of field rivets. It becomes a necessity where a shallow floor is demanded.

This often occurs. The shelf angle is then omitted, Fig. 77*d*. There is evidently nothing to be gained by reducing d, but b may be cut down to b', the smallest possible value of which is the thickness of the angle; yet the latter should not be considered as carrying any load.

There is no way of computing the size of the connection angles. However, material less than $\frac{7}{16}''$ thick should not be used and $\frac{1}{2}''$ is better. Smallest allowable leg is $4''$; if this will not hold enough rivets, use a $6''$ leg.

Stringers at Abutments

Two methods of constructing bridge at ends are shown in Figs. 77*e* and 77*f*. The latter costs more for the steel but

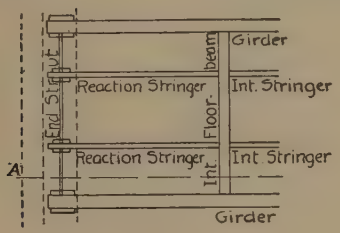


Fig. 77*e*.

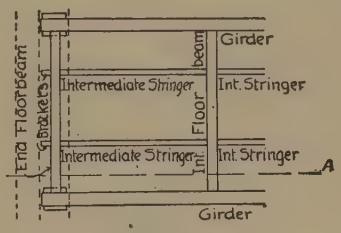
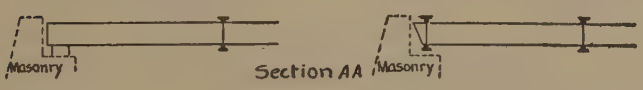


Fig. 77*f*.



lessens masonry. For the former, the stringers have very simple shoes of types 1 and 2, Art. 61, at fixed and free ends respectively. Use two plates of a size which computations show to be necessary and of $\frac{3}{4}''$ thickness. At expansion ends they should be planed where in contact and holes slotted to allow for motion. Use bolts $\frac{3}{4}''$ diameter by $12''$ long or thereabouts. To stiffen the I-beams at their ends, a channel strut of a depth not less than one-half that of the stringers should be employed. This "end strut" should frame in between stringers, also between stringers and girder. It thus becomes continuous for the width of the bridge and serves to transmit the wind stresses as well.

For the other method, no end strut is necessary, as the end

floor beam acts as such. Small brackets or short stringers are then employed to carry the ties in the gap between end floor beam and masonry. See Figs. 77*f* and 90*a*.

Prob. 77. A 100′ through span railroad bridge has $a=7.0$ and $d=18.0$. It is figured for 6000 lbs. per lin.ft. per track. What should be the panel length?

Art. 78. Floor Beams for Open Floor Railroad Bridges

In general a floor beam may be either a small riveted truss, an I-beam, or a built-up girder. For this case, the latter with web, four angles, and cover plates extending as far as required

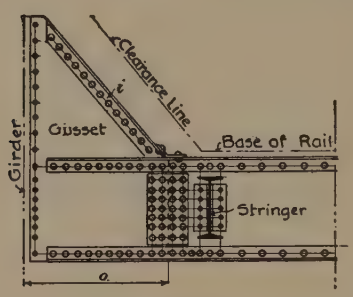


FIG. 78*a*.

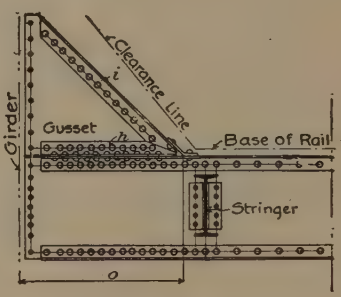


FIG. 78*b*.

Two Methods of Bracing Top Flange of Half Through Bridges.

is well-nigh universal. Many engineers prefer to keep cover plates narrow, say 9 or 10″ in width. To do this, they use a 4″×4″ flange angle or a 6″×4″ with shorter leg horizontal. The idea is to leave less space between ties, thus decreasing danger of " bunching " in case of derailment.

The principal office of the floor beam is to carry the loads from the stringers to the girders. It also acts as a strut in the lateral system and serves to brace the top chord. We have considered in Art. 76 the stresses which are caused thereby. Figs. 78*a* and 78*b* show two methods of accomplishing this.

There is little to choose between them. In either case, the " gusset," as the triangular part is termed, should extend

to the top of the girder in order to brace it effectively. *o* should be made as large as connections and clearance diagram will allow. For ease of fabrication, thickness of web of gusset is best made the same as that of the floor beam. Two angles are needed for *i* and *h*; the minimum size may be used, but it is better, especially for heavy spans, to specify $4'' \times 4'' \times \frac{1}{2}''$ or larger.

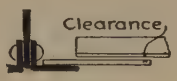


Fig. 78*c*. Bottom Flanges of Floor Beam and Girder.

The floor beam should connect to girder by two stiffeners. If both these were shop riveted to girder, it would constitute a very difficult entering joint. To avoid this, field rivet one stiffener to girder and ship bolted to same. It may then be loosened temporarily and oor beam swung into position.

The bottom of the floor beam is raised above the top of the horizontal leg of the lower flange angles by an amount equal to the thickness of the lateral plates plus $\frac{1}{16}$ to $\frac{1}{8}''$ for clearance.

Prob. 78. Design stringers and floor beams for a double-track bridge having girders 30′ c. to c. and panel lengths of 20 feet. Loading Cooper's E 50. Stringers 6′ 6″ c. to c. Use allowable stresses of Art. 50 for medium steel.

Art. 79. Plate Girder Flanges

Construction is much alike for all types of bridges.

The box girder, Fig. 54*f*, is so rare for this location that we shall not attempt to consider it here. It is used to carry heavy loads in a limited depth.

A girder may be divided into two parts: the *web* with its fittings, and the *flange*. The necessary area of the latter is determined by computation. As already explained in Part I, we design weaker flange and make both alike, except that sometimes one top cover plate is carried way across to better protect from rust. If properly supported laterally, compression flange is usually stronger.

In selecting material to make up this flange area which may run as high as 250 sq.in., thickness ought not to exceed

diameter of rivet to avoid drilling from the solid, Art. 44; if sub-punching and reaming are specified, which frequently occurs, steel should be $\frac{1}{8}''$ thinner. To prevent rapid deterioration from rust, $\frac{3}{8}''$ is the minimum; we are thus limited to material between $\frac{3}{8}$ and $\frac{3}{4}$ or $\frac{7}{8}''$. ($\frac{7}{8}''$ is the customary size of rivet on railroad work.) There is a slight tendency to use the upper limit on long spans. The width of the cover plate should never be less than the out to out of flange angles with an allowance for overrun and building up. $12''$ plate over $5''$ legs, $14''$ over $6''$, and $18''$ over $8''$ is customary practice except for Type *B*, where width may be increased $2''$ unless edge distances become too great, Art. 50, *e* 1.

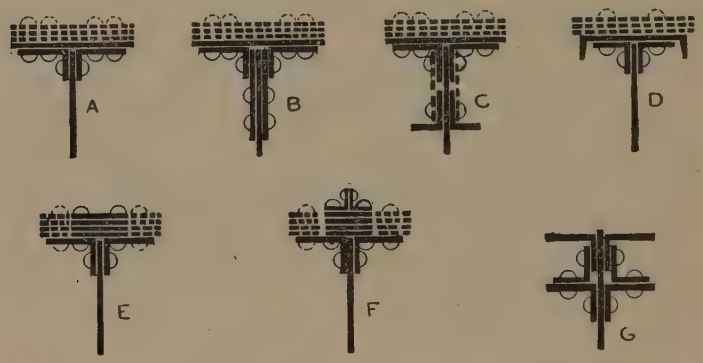


FIG. 79*a*.—Flange Sections.

To prevent the use of disproportionately light angles for a heavy flange, it is specified that one-third of flange area shall be in angles or side plates, Art. 50, *d* 13. Up to 50 sq.in. required area, Type *A* may be used with $6''\times6''$ angles. $8''\times8''$ might be employed, but they cost extra. However, they should be specified on spans exceeding 90 feet. From 50 sq.in. up, the writer prefers Type *B*. However, *C* and *D* are occasionally seen. For the largest bridges, $8''$ angles will be found necessary, $6''\times4''$, $6''\times6''$, $8''\times6''$, and $8''\times8''$ are customary, the longer leg being horizontal. Always ascertain if the vertical leg will hold the necessary rivets. Types *E*, *F*, and *G* are designed with a view of lessening cost of maintenance. Ties are ordinarily notched as shown in Fig. 79*b*.

The depth, *d*, at a point where the flange is heaviest is the specified minimum. As the required area diminishes, the total thickness of flange plates decreases and depth of tie must be increased, thus adding largely to cost. Types *E* and *F* eliminate this varying depth, while Type *G* in addition simplifies cutting for rivets, a very simple notch as seen in Fig. 79*c* being sufficient. Type *C* acts similarly if flange plates are placed vertically upon the angles. Grooving may be obviated by countersinking vertical rivets, but this is rather unusual.

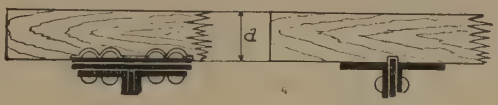


FIG. 79*b*. FIG. 79*c*.

Methods of Dapping Ties.

For through and half through bridges, the upper ends are often rounded for appearance's sake with a radius of about two feet, Fig. 79*d*. We have already noticed that angles and one top cover plate were continued to the end. In this case, they are bent and form the end stiffener. They should be spliced on top within a few feet of the point of curvature. There are

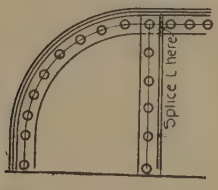


FIG. 79*d*.

Rounded End.

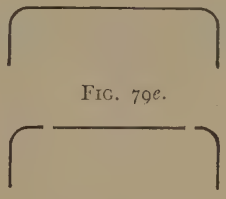


FIG. 79*e*.

FIG. 79*f*.

Splicing of Bent Angle.

two reasons for this: the difficulty of bending long pieces and the inaccuracy of work if there are two angles in the same shape. To explain further, in Fig. 79*e*, it is hard to bend the ends so that they will be the proper distance apart. In Fig. 79*f*, inaccuracy can be eliminated by ordering a little extra material, shearing off, and then punching to a templet.

Flange Splices

(*a*) If possible avoid two splices at the same point. This applies with equal force to one in web and one in flange.

(*b*) Sufficient distance should be left between them such that the member cut is fully spliced before next one is begun.

(*c*) Endeavor to keep rivets short and splice material as close to cut as possible. Some engineers make a deduction from value of rivet for each intervening plate.

(*d*) Splices are planes of weakness and are better located at points where total stresses are small and unit stresses are below permissible values.

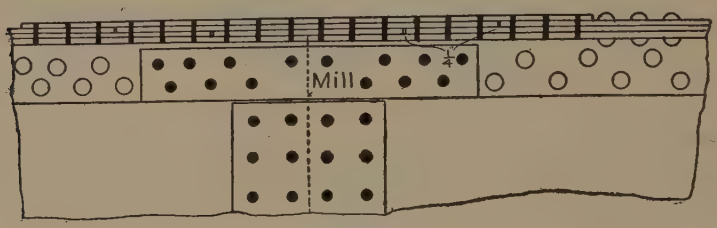


Fig. 79*g*.

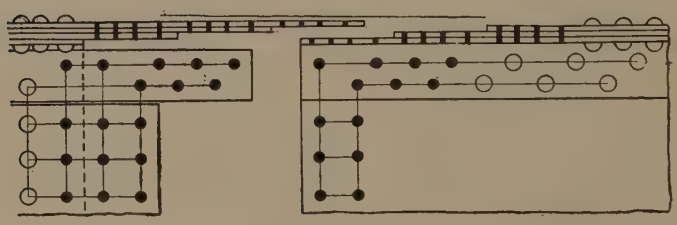


Fig. 79*h*.

Field Splices in Plate Girders.

Plate girders for this country need not be riveted together in the field. In case suitable shipping facilities are lacking, would prefer a riveted truss. Structural and railroad companies possess equipment for fabricating, shipping, and erecting any ordinary spans full size. Where field splices are required, it will be found necessary to largely ignore (*a*). There are two

methods: the first is to cut girder square off and put in sufficient material to fully splice, Fig. 79*g*. The other is to step off plates as shown in Fig. 79*h*. To avoid projecting pieces in shipment (Art. 48) and entering joints (Fig. 49*m*), web and flange angle splices are better if shipped loose.

Modification of these two methods are employed for *shop splices*. Splicing of flange plates is illustrated diagrammatically in Fig. 79*i*. If we compute the required flange areas at dif-

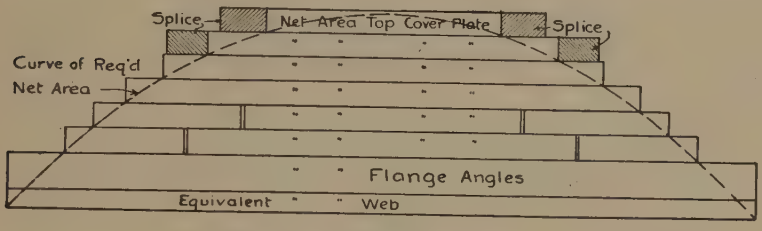


FIG. 79*i*.—Flange Diagram.

ferent points of a girder, we may construct curve as shown. Suppose now we plot horizontal lines, the distance between them representing areas furnished by web (equivalent area, see Art. 54,*f*), angles, and cover plates taken in order, we readily ascertain cut-off points for plates. This is the procedure for the *intermediate method*. The *approximate method* is similar excepting the omission of the equivalent area of the web. In the *exact method*, the curve represents *resisting moment* (I/c) while the horizontal lines show the resisting moment for successive combinations as the various cover plates are omitted, see Fig. 89*c*. In practice, they usually extend a few inches beyond points as ascertained above, in order to allow room for enough rivets to carry an appreciable stress. The method of splicing consists in locating joint where another plate may be prolonged far enough beyond its theoretic cut off point, to form a complete splice. A more economical way of regarding it, is to consider as shown in Fig. 79*j*. That is, we assume stress from 1 *a* is transferred into 5 and 1 *b* takes

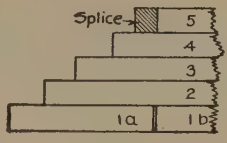


FIG. 79*j*.
Method of Splicing
Flange Plates.

up the increment of stress. This saves 50% of the additional material.

It is customary to splice the two angles in one flange as shown diagrammatically in Fig. 79*k*. The author believes

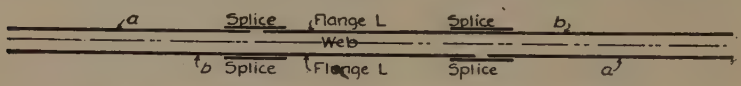


FIG. 79*k*.—Customary Arrangement of Flange Splices.

that the eccentricity induced thereby is as objectionable as carefully splicing two angles at the same point as shown in Fig. 79*l*.

There are three methods of splicing an angle:

(1) *Rolled splice angles, Fig.* 23*c*.
(2) *Fabricated splice angles, Fig.* 79*m*.
(3) *Splice bars, Fig.* 79*n*.

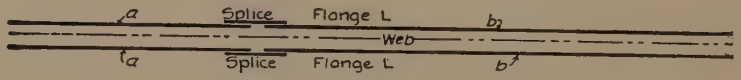


FIG. 79*l*.—Proposed Arrangement of Flange Splices.

(1) is very little used on account of the difficulty of securing special sections. In (2), a standard section has the corner *a* milled down and the ends sheared off to fit. After this treatment, it should still contain as much net area as the angle which

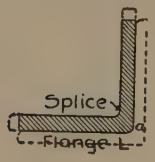


FIG. 79*m*.
Fabricated Angle
Flange Angle Splice.

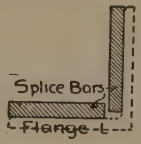


FIG. 79*n*.
Splice Bars.

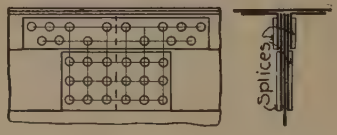


FIG. 79*o*.
Splice for Side Plates.

it splices. (3) is the best design on account of the simplicity of arrangement.

Side plates are spliced in a general manner similar to angles. Of the three types given above, the latter with splice bars as shown in Fig. 79*o* is usual.

Prob. 79*a*. A plate girder of type *G*, Fig. 79*a*, has $80\times\frac{1}{2}''$ web plates; outer and inner angles, $6''\times4''\times\frac{1}{2}''$ with longer leg horizontal; intermediate angles, $4''\times4''\times\frac{1}{2}''$. Back to back of angles, $69\frac{1}{2}''$ and $79\frac{1}{2}''$. Rivets are $\frac{7}{8}''$ diameter. Compute moment of resistance.

Prob. 79*b*. For the same data, determine by the exact method, maximum spacing of each row of rivets for a shear of 80,000 lbs. and a vertical load of 800 lbs. per lin.in. Allowable shear 7500, bearing 15,000 lbs. per sq.in.

Art. 80. The Plate Girder Web

The web is generally uniform in depth and thickness, the latter often the minimum allowable. However, it may be thickened to carry heavy shears, to increase rivet and stiffener spacing, or perhaps to eliminate the latter altogether. Where an additional thickness is necessary at the ends, it is best furnished by two plates, riveted on between the angles as shown in Fig. 80*a*.

Stiffeners

These are placed on the webs at intervals, usually in pairs as seen in Fig. 80*a*. They are fitted tightly against flange angles, see Figs. 37*v*, 37*w*, and 37*y*. They may be "crimped" (bent as shown in Fig. 42*b*) so as to lay flat against web between angles, or "filled," that is, have a filler the same width as the leg of the stiffener parallel to the web, of a length equal to the distance in the clear between flange angles and of a thickness equal to that of the flange angles. The filled stiffeners should be employed where there is heavy stress or an important connection; elsewhere either may be used. Crimped stiffeners save material but cost more per pound. Fabricating companies are likely to prefer them in "lump sum" contracts, rather than "pound price" jobs.

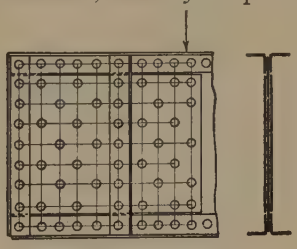


Fig. 80*a*.
Reinforcement of Web at End of Girder.

The thickness of the intermediate stiffener angles is usually the smallest allowable; the outstanding leg is made as long as possible without projecting from the flange angles which they support; the other leg about as small as riveting will allow. The end stiffeners are made the same except that thickness is often increased. They may be computed as columns to carry the maximum end shear. Thus for $6'' \times 6''$ flange angles and $\frac{7}{8}''$ rivets use $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ stiffeners. Art. 86 will consider in detail the theory of stiffener spacing.

Splicing Webs

Except for very small plate girders, the extreme lengths of rolled plate are such that the web must be spliced. If figured to carry shear only, it must be spliced for shear; if computed for both shear and moment (exact and intermediate methods, Art. 54), it must be spliced for moment, since the shear for plate girders at splices is, compared with the moment, usually negligible. Experience in actual service has demonstrated beyond a doubt that web should be spliced for moment, thus showing correctness of exact theory.

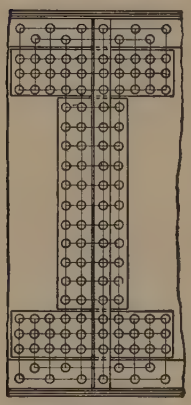


Fig. 80*b*.
An Economical Splice for Carrying Moment.

In splicing for shear, we should employ not less than two rows of rivets on each side of the joint with splice plates as thick as the angles. In case this furnishes too much or too little area, thickness may be altered and fillers employed, but the method above outlined gives simple details and usually satisfactory strength. Shear at splice divided by value of rivet, usually in bearing on the web determines number necessary.

Splicing for moment will require more rivets. Since stress in web is near allowable limit, we sometimes simply splice net area of web without regard to actual moment. Use methods of Art. 59 to design plates. See also Art. 89 for the computation of splice in a large plate girder. The farther the rivets are from the neutral axis, the more effective they become, hence joint is sometimes designed as shown in Fig. 80*b*.

Prob. 80*a*. Disregarding rivets in flange angles, find capacity of joint in Fig. 80*b* in terms of rivet value, *R*. Spacing, 3″.

Prob. 80*b*. What is the percentage gain in efficiency of arrangement like Fig. 80*b* over an equal number of rivets with the same three-inch spacing in full vertical rows?

Art. 81. Shoes and Bracing for Plate Girders

Art. 61 describes shoes in general and it will suffice in the main for plate girders. It may be noted, however, that if the bridge is on a grade it is cheaper to bevel masonry plates rather

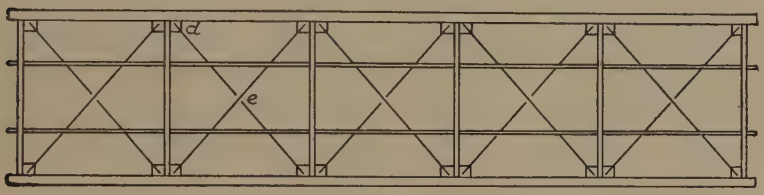


FIG. 81*a*.

Arrangement of Bracing in a Half Through Plate Girder.

than sole plates. However, many engineers object and prefer to bevel the latter. For further examples of shoes and methods of computations, see Arts. 89 and 93.

Fig. 81*a* shows a line diagram of the bracing. Each member consists of angles, usually single, field riveted to connection plates, likewise field riveted to top of lower flange angles of girders and bottom of bottom flange angles of floor beams. The latter act as struts and, in connection with the gusset plates, serve as transverse bracing. See Art. 78.

If more than four rivets are required for connections of a diagonal of one shape, *lug angles*, Fig. 81*b*, should be employed. This practical rule will be found to agree roughly with that given in Art. 50, *d* 5.

There are two ways of arranging angles at their intersection: (*1*) turning them opposite ways and inserting a filler, Fig. 76*e*; or (*2*) making one angle continuous while other is spliced,

Fig. 81*c*. Care must be taken that the net area on line *aa* is sufficient. One disadvantage of the former is that unless flange is heavy, the laterals will lessen clearance and appear unsightly. (*2*) is the usual method.

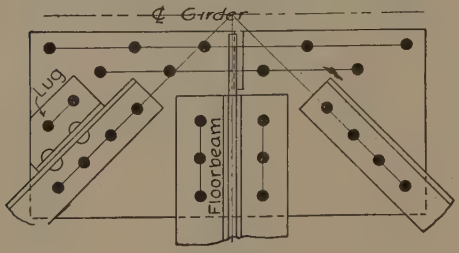


Fig. 81*b*.
Detail of Joint at *d*, Fig. 81*a*.

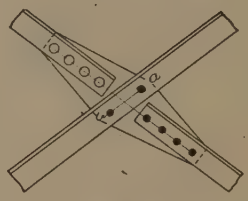


Fig. 81*c*.
Detail at Intersection of Diagonals at *e*. Fig. 81*a*.

If stringers connect to floor beam as shown in Figs. 77*b* and 77*c*, then second type should be employed and fastened to stringers as shown in Fig. 81*d*. This stiffens stringers, decreases stresses in laterals due to own weight, and tends to render angles capable of carrying some compression.

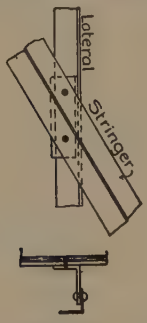


Fig. 81*d*.
One Connection of Lateral to Stringer.

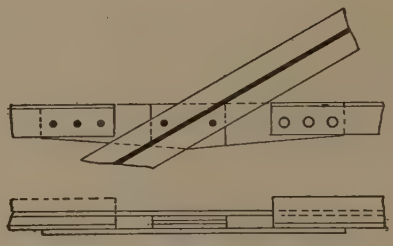


Fig. 81*e*.
Another Connection of Lateral with Stringer.

If we have conditions shown in Fig. 77*d*, to deal with, we may employ three methods:

(*a*) Use (*2*), turn angles up, splice both laterals by plates at stringers, using filler plates if necessary, Fig. 81*e*.

(*b*) Use (*2*), turn angles down, employing fillers at stringers.

(*c*) Use (*1*), splicing the upturned angle at stringers.

(*a*) is customary, since, for reasons already explained, it is inadvisable to turn angles down.

If without end floor beams, the compression which they would otherwise carry should be taken by a strut, usually a channel not quite as deep as the stringers and field riveted to them by single angles. It may be fastened to the girders by field rivets through stiffeners.

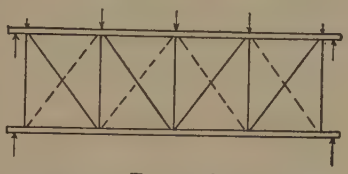


Fig. 81*f*.
Arrangement of Laterals Keeping End Strut away from Shoe.

There is often trouble in driving field rivets which connect end lateral plate to shoe. To avoid this:

(*A*) Intersection points for bracing may be taken far enough towards center of the span to allow underneath room sufficient for properly driving rivets. This makes the lateral system act as shown in Fig. 81*f*. We scarcely need to say that this is poor design.

Fig. 81*g*.—Half Through Plate Girder Bridge Carrying Baltimore & Ohio R. R. over Street near Homestead, Pa.

(*B*) Rivet lateral plate on before girder is lowered into position. This adds considerable trouble and expense in erection.

(*C*) Use tap bolts. This is bad from the standpoint of the shop.

(*D*) Use a pin or raised bearing which will allow room underneath to drive rivets readily. While this adds to cost of steel, it makes a more accessible bearing and lessens masonry.

Prob. 81. A $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$ angle diagonal standing thus L, braces a bridge of 7 panels @ 14 feet. Girders are 16′ c. to c., stringers 7′. What is stress due to own weight (*a*) when free, (*b*) when supported by stringers? (*c*) Answer above questions with a tension of 30,000 lbs., using second method of Art. 55.

Art. 82. Solid Floor Plate Girder Bridges for Railroads

We have already treated solid floors in Art. 72. Here we will consider their adaptation to plate girder bridges for railroads.

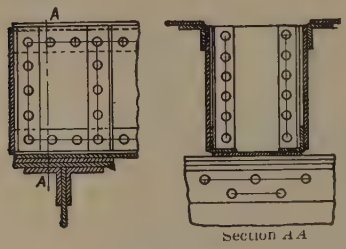


Fig. 82*a*.—Type 1.

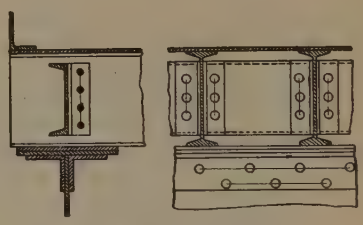


Fig. 82*b*.—Type 2 or 3.

Solid Floors for Deck Plate Girders.

The locations which demand solid floor bridges, that is, those over streets with large traffic, usually have surroundings which make a minimum clearance and hence through bridges, imperative. But a deck structure should be employed in places where clearance is not important. The entire bridge below the floor is then made like an ordinary deck girder except that upper lateral bracing is unnecessary and that field rivets are driven through upper flanges to fasten solid floor thereto. See Figs. 82*a* and 82*b*.

In through bridges, the floor is attached to plate girders just to clear lower flange, by methods considered in Art. 72. It takes the place of stringers, floor beams, and lower laterals in a half through structure. Like the latter, the sway bracing consists of gusset plates fastened to the stiffeners and extending to the top of the girders. Methods of designing and detailing are similar except that in erection the gussets may be shop riveted to girders or shipped loose. If attached to floor sections, shipment is made difficult.

Fig. 82*c*.—Half Through Plate Girder Bridge with Solid Floor Carrying Tracks of Pennsylvania Railroad over Kelly Street, Pittsburgh, Pa.

Art. 83. Railroad Bridges on Grades, Curves and Skews

Bridges on Grade

Few bridges, either for railroad or highway, have grades exceeding 5%; for such it will cause no harm in *design* if we consider it as horizontal and of a length s, Fig. 83*a*. It is then detailed exactly the same as a horizontal bridge whose span is l, except that masonry plates should be bevelled to the grade.* This is shown, exaggerated as to elevation, in Fig. 83*a*.

* This is to make both ends of a girder alike. There is more theoretic stability, however, with bevelled sole plates.

Bridges on Curves

There are three reasons why a bridge on a curve differs from one on a tangent:

(*1*) *The Superelevation of Track.* That is, the outer rail is raised enough so that resultant force at maximum speed is per-

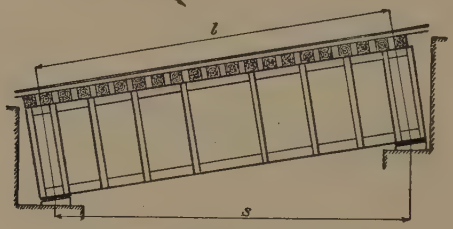


Fig. 83*a*.—Bridge on a Grade.

pendicular to track. This is usually accomplished as shown in Figs. 71*c* and 71*d*, and steel is not altered thereby. Occasionally, the shoes are arranged to give the sidewise tilt necessary for proper superelevation of rails.

(*2*) *The Centrifugal Force.* Structure must be computed

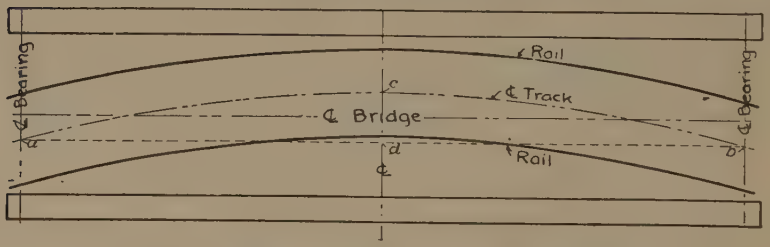


Fig. 83*b*.—Spacing of Girders on a Curved Bridge.

with loads at rail either vertical (train at rest) or perpendicular to track (full speed), that one being chosen for a given part which will produce maximum stress. The portions which are most affected are the bracing and shoes. These must be increased in strength to correspond with the forces.

(*3*) *Varying Positions of Rails.* To locate plate girders in a deck structure, proceed as follows, Fig. 83*b*: Let line of

supports intersect center line of track at *a* and *b* and let *cd* be the ordinate at the middle. Draw center lines of girders parallel to chord, equidistant from the middle point of *cd*, and so as to clear gage line of rails by at least one foot.

Stringers in through bridges may be similarly arranged,

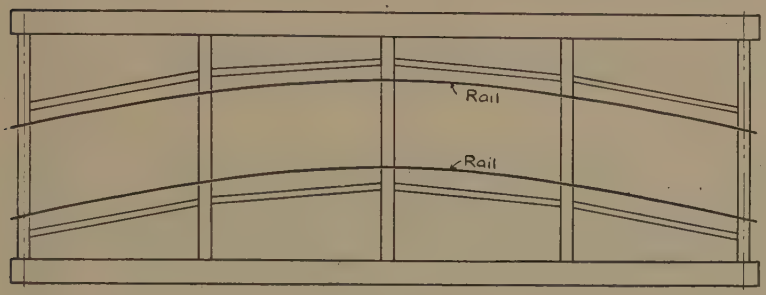


FIG. 83*c*.

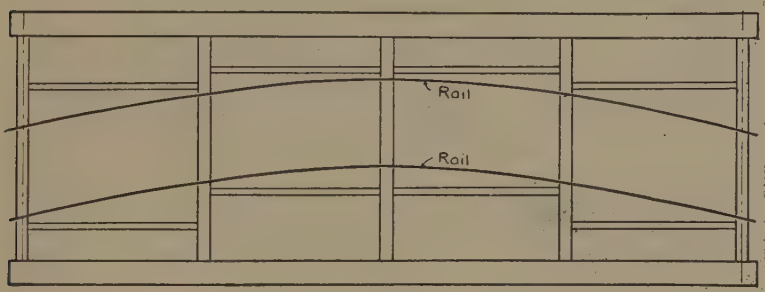


FIG. 83*d*.

Methods for Locating Stringers in Half Through Bridges.

Fig. 83*c*. However, this involves skew stringers, and bent angles with correspondingly increased costs in shop and drawing room. To avoid this, we may design as in Fig. 83*d*, care being still taken to keep stringers well outside line of rails. Connection to floor beams must then be carefully studied, as field rivets must match or clear. Fig. 83*d* is probably the better arrangement for flat curves. Girders for through and half through bridges are located as in deck structures except

that additional width is needed for clearance. Solid floor bridges for curves obviously need differ from those on tangent only by the slightly greater distances required between girders.

Bridges on a Skew

Where the angle between longitudinal center line of bridge and abutment is not 90 degrees, the structure is said to be on

Fig. 83*e*.

a skew, the amount of the deviation from a right angle being known as the angle of skew.

Like curve bridges, they will probably have a great deal of bent work and there will be less duplication of pieces than in an ordinary structure. Moreover, the computation of stresses and lengths is quite tedious. Hence avoid skew bridges if possible. It is frequently better to lengthen the bridge some-

what as shown in Fig. 83*g*, rather than pay for the increased shop work of type seen in Fig. 83*f*. Sometimes where appearances or the necessity of preventing eddy currents in the stream

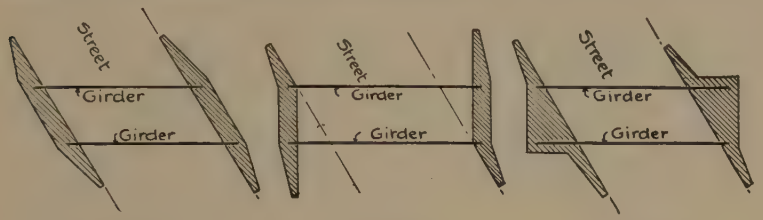


FIG. 83*f*. FIG. 83*g*. FIG. 83*h*.

Arrangements for Skew Bridges.

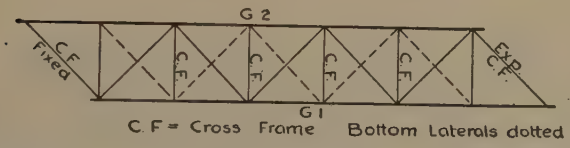


FIG. 83*i*.—Arrangement of Skew, Deck Open Floor Plate Girder.

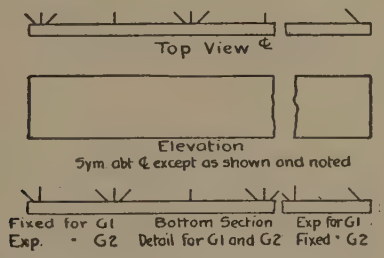


FIG. 83*j*.—Method of Detailing above Bridge.

FIG. 83*k*.—Arrangement of Skew, Half Through Plate Girder.

compel abutments to be placed on a skew, bridge may still be made square as shown in Fig. 83*h*; in all cases, the question of least ultimate cost must decide.

Figs. 83*i*, 83*k*, and 83*l* show respectively proper methods of arranging skew bridges for deck open floor, half through open floor, and through solid floor. The former is designed

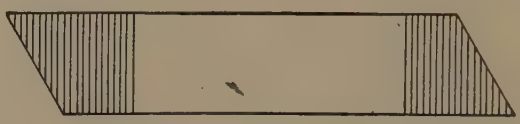


FIG. 83*l*.—Arrangement of Skew Solid Floor Plate Girder.

to admit of detailing as represented in Fig. 83*j*. A like simplicity may be effected for the other types, thus promoting economy in detailing and templet making.

FIG. 83*m*.

Prob. 83. What should be the least distance between deck girders of 100 foot span for a standard gage track on a 5° curve?

Art. 84. Highway Plate Girders

The principal points of difference which we note in these bridges as distinguished from railroads, are the *lighter loads*, the *less stringent specifications*, and the *somewhat larger widths*. Deck bridges have very little advantage over the half through type and the latter are the more common structures, Fig. 84*a*. Occasionally the stringers are omitted and the floor beams placed close enough together so that the flooring will not be overstressed.

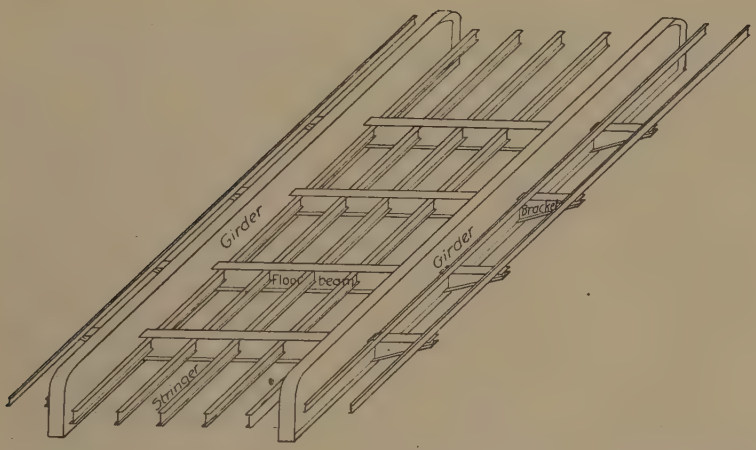


FIG. 84*a*.—Typical Highway Plate Girder Bridge. End struts and laterals not shown.

The latter is designed as explained in Arts. 73 and 74. Stringers are usually framed in for roadway and set on top of bracket. They may be either of steel or wood. In the former case, they are made much as in I-beam spans except for their connection to the floor beams. Where placed on top, the beam is simply riveted to flange, fillers being possibly interposed to bring to the required level. If framed in, the customary two-angle connection is employed. Shelf angles are desirable for erection purposes; in the best designs they are supplemented by stiffeners underneath, Fig. 84*b*. In a

case like this, the author would prefer to figure shelf and stiffener angles alone to carry the load and have it *erected to correspond to that assumption.* In case stiffener is omitted, the probable concentration of load on the outside edge of the angle will cause a fiber stress of,

$$S=\frac{6M}{bd^2}=\frac{6Vl}{bd^2}, \text{ Figs. 84}b \text{ and } 84d.$$

If S be taken as 12,000 lbs. per sq.in., we have,

$$d=.0224\sqrt{\frac{Vl}{b}}, \text{ everything in pounds and inches.}$$

If $V=2500$ #, $l=2\frac{1}{2}''$ and $b=9''$, we find that $d=\frac{5}{8}''$.

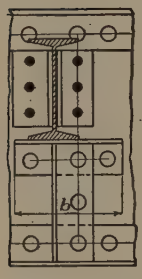


Fig. 84*b*.
Steel Stringer.

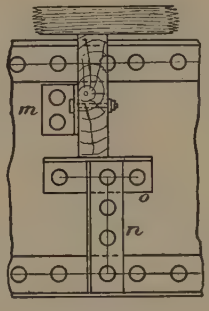


Fig. 84*c*.
Wooden Stringer.

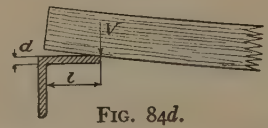


Fig. 84*d*.
Action of a Load on a Shelf Angle.

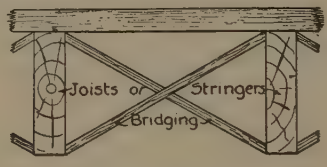


Fig. 84*e*.
Bridging.

Connection to Floor Beams.

The best design for wooden stringers framing in is shown in Fig. 84*c*. Sometimes the angle *o* is made continuous. Often either *m* or *n* or both are omitted. At abutments the stringers should be toenailed to a timber, bolted to masonry, Fig. 84*f*. The theoretic economy of a rectangular beam increases with the ratio of depth to thickness. Practically, however, this ratio must not be allowed to exceed *seven*, on account of the

tendency of deeper joists to buckle. Beams must be firmly held at the ends, as in Fig. 84*c*, and braced by rows of $2'' \times 4''$ bridging, Fig. 84*e*, at intervals not exceeding 10 feet. Unless so braced, breadth must not be less than *one-fourth* the depth.

To form variations in height transversely, we may alter nailing pieces or change steel work to give stringers the necessary difference of elevation. It is a matter of costs in either case.

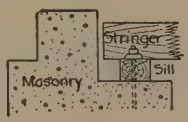


Fig. 84*f*.
Bearing Wooden Stringers.

Floor Beams

These may be made of an I-beam, rolled or built up, the latter being perhaps the more common. As in railroad bridges, gussets to the top flange should be provided. At connection to the girder, only one stiffener need be employed except in heavy bridges. The stresses which these gussets carry and the methods for computation are considered in Art. 76. However, this part is often slighted, the strength of the riveted connection between floor beams and girders being assumed to carry the stresses.

Brackets

These are usually located outside of the girder and opposite the floor beams. They are generally made in the trapezoidal

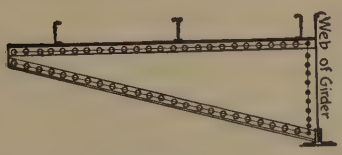


Fig. 84*g*.
Solid Web Floor Beam Bracket.

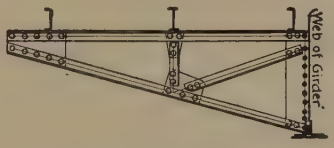


Fig. 84*h*.
Trussed Floor Beam Bracket.

form shown; thus built, they are economical of material without sensibly increasing shop costs. There is little to choose between the two types; Fig. 84*g* involves more material at a lower shop cost per pound. It is figured as a plate girder of

variable depth. On the other hand, Fig. 84*h* is computed as a truss. One trouble is to so design connection at girder that it will not cause the objectionable tensile stresses on the heads of the rivets. The usual method is to field rivet to an outstanding leg of a stiffener, thus bringing the tension upon the shop rivets. In a half through plate girder, a bar may be passed through a slotted hole in web of girder as seen in Fig. 84*i*. This is a good design for bracket, but it is expensive in shop work and must weaken web. If clearance underneath will allow, girder may be lowered until this splice plate can pass over the top flange. Sometimes floor beams are placed directly on top of girders. Great care must then be taken to see that they are efficiently braced. Under these circumstances, the girders do not have to be placed under the curb, but may be so located as to promote economy of material. See Art. 105.

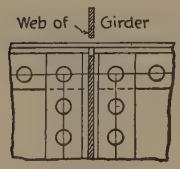


FIG. 84*i*.
Connection of Floor Beam Bracket to Floor Beam.

LATERALS

The distance between floor beams is determined by economical considerations, 12 to 15 feet being common. Laterals may be arranged as shown in Fig. 84*j* or 84*k*, the latter being

FIG. 84*j*.

FIG. 84*k*.

Arrangements of Lateral Bracing.

preferred for the greater widths. One angle designed as a tension member is usual, although the objectionable adjustable rod still survives. Lack of care in securing central connections is often seen.

Girders and shoes are much like those for railroad bridges.

SOLID FLOOR HIGHWAY BRIDGES

The usual type of solid floor is the buckled plate resting on steel I-beam stringers. The latter are designed and sup-

ported as in plank-floor bridges except that they are spaced much farther apart, and, in common with the rest of the steel, must be heavier to carry the increased dead load. Trough plates from girder to girder and solid plate resting on floor beams placed close together also occur. A solid floor is considered to be effective bracing in its own plane.

There is seldom any need for locating highway bridges on curves. Bridges which are skewed or on a grade are handled in a manner analogous to the similar cases for railroads.

Bridges for Electric Cars

When bridges are built for electric cars alone, they become, for all practical purposes, light railroad bridges. Permissible stresses and other specifications will vary somewhat with loads and speeds; the heavy *express interurban cars* need the same care in design as *railroads;* on some highway bridges, the actual loads due to the slowly moving street cars employed may be such that no special provision is required. However, the probability of changes in the future should always be borne in mind.

Prob. 84. Design brackets in Figs. 84*g* and 84*h*, and compare weights when spacing of joists is 4′ 6″; over all length 11′ 0″; depth at girder 3′ 0″; load on center joist 8000#; half load at other beams; allowable stresses for medium steel as given in Art. 92.

Art. 85. Absolute Maximum Moment

Each section of a beam subjected to the action of moving loads a fixed distance apart has its maximum moment. When we wish to designate that one of these moments which has the highest value, we use the term, " absolute maximum moment."

This subject more properly belongs in " Stresses " or " Mechanics," but it is often slighted and is here given in full.

Let a certain portion of the loading, *W*, Fig. 85*a*, having

a center of gravity distant $L-x+a$ from right abutment, be assumed to be on the span L, where x represents the distance to the point of maximum moment where the shear passes through zero, P_5. Then moment, M, at P_5 equals,

$$\frac{W(L-x+a)}{L}x - m,$$

where m represents the moment of the preceding loads about P_5. Differentiating for a maximum,

$$\frac{W}{L}(L-2x+a)=0 \quad \text{or} \quad x=\frac{L}{2}+\frac{a}{2},$$

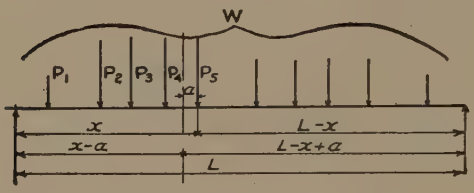


FIG. 85*a*.—Moving Loads on a Beam.

or, in words, *Place the loads assumed to be on the span so that the center of the span comes half way between center of gravity and that load where the shear passes through zero.*

In practice we proceed as follows:

(*1*) *Assume a number of consecutive loads upon the span.* The heavier loads, those spaced closely, and a combination which brings them near the center of the span is most likely to give the absolute maximum. A great deal of time may be saved by eliminating those loadings which obviously will not produce the value sought. Usually, but not always, the absolute maximum moment occurs at the same load which produces the maximum moment at the center. This loading, it will be remembered, has a load at the center so located that, if considered to left, there is more load on that half; if to right, balance is on the right side.

(*2*) *Compute center of gravity.* $\bar{x}=\frac{\Sigma Px}{\Sigma P}$. ($\Sigma Px$ is the moment

in a moment diagram for locomotive loads, while ΣP is the shear.)

(3) *Assume load at which shear equals zero.*

(4) *Place loads so that center of span comes half way between center of gravity and load where the shear was assumed to pass through zero.*

(5) *Test to see if loads are on span as assumed.* If not, begin again, revising (1).

(6) *Test to see if shear passes through zero at load assumed.* If it does not, revise (3) and proceed from that point.

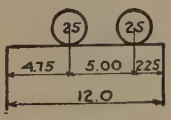
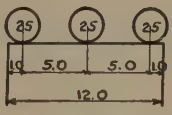
(7) If preceding assumptions be correct, compute moment at point where the shear equals zero.

(8) Test as many reasonable assumptions as may be necessary to make sure that the maximum maximorum has been obtained.

The maximum moment at center will generally be close enough for spans exceeding 60 feet. For a single rail of Cooper's E 50, the maximum moments at the center of 10 and 60 feet spans are respectively 62.5 and 1617 kip feet. The corresponding absolute maximum moments are 70.3 and 1623 kip feet.

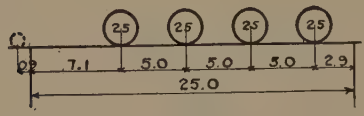
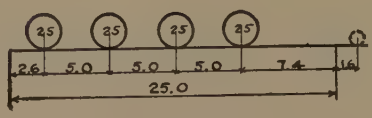
We will now compute absolute maximum moments for one rail of Cooper's E 50, as an illustration of proper procedure. Let the loads be numbered consecutively, P_1, P_2, etc.

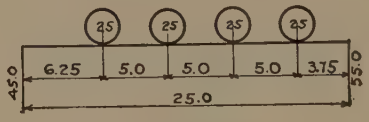
SPAN 12 FEET

(1) Assume P_2 and P_3	Assume P_2, P_3, and P_4
(2) Center of gravity half way between	At P_3
(3) Shear equals zero at P_2 or P_3	At P_3
(4)  FIG. 85*b*.	 FIG. 85*c*.
(5) Loads on span as assumed	Loads on span as assumed
(6) Shears pass through zero at loads assumed	

(7) $M = 94.0$ kip feet	$M = 100$ kip feet Abs. Max. Mom.
(8) One load would be like three except for additional outside wheels. Four cannot be placed upon the span.	

SPAN 25 FEET

(1) Assume P_1 to P_5 incl.	Assume P_2 to P_6 incl.
(2) $\bar{x} = 1037/112.5 = 9.2$ from P_5	$\bar{x} = 1650/116.25 = 14.2$ from P_6
(3) Shear equals zero at P_3	Shear equals zero at P_4
(4)  FIG. 85*d*.	 FIG. 85*e*.
(5) P_1 comes off the span, revise	P_6 comes off the span, revise

(1) Assume P_2 to P_5 inclusive
(2) Center of gravity half way between P_3 and P_4
(3) Shear equals zero at either P_3 or P_4
(4)  FIG. 85*f*.
(5) Loads are on span as assumed
(6) Shear passes through zero as assumed
(7) $M = 381.25$ kip feet, Abs. Max. Mom.
(8) No other loading can give more moment

Prob. 85. Check values of absolute maximum moment fo Cooper's E 50 on a span of 60 feet as given above.

Art. 86. Theory of Stiffener Spacing

In Art. 50 of Vol. I, we gave a "rule of thumb" method for spacing stiffeners. This is employed a great deal in design. We will now consider the theory.

Mechanics teaches us that a vertical unit shear is accompanied by an horizontal shearing unit stress of equal intensity, the combination of the two resulting in a tension and compression at 90° with one another and 45° with the horizontal, and having an intensity equal to either shear.

Let S_c represent allowable compressive unit stress for short strut;
C = coefficient of slenderness ratio in straight line column formula;
V = total shear;
t = thickness of web;
d = depth of web, Fig. 86*a*;
s = spacing of stiffeners.

The compression on a diagonal line of length l may equal the allowable

$$S_c(1-Cl/\rho)=S_c(1-5Cs/t).$$

As already noted, mechanics has shown that this diagonal compressive unit stress equals the shearing unit stress equals V/dt. Hence,

$$V/dt=S_c(1-5Cs/t) \quad \text{or} \quad V=dS_c(t-5Cs).$$

Due to the fixed ends, the support of flanges and stiffeners and of the tension at right angles, C may be safely taken as low as .001 and formula becomes:

$$V=dS_c(t-.005s)$$

also

$$s=200(t-V/S_cd)$$

or

$$t=\frac{V}{dS_c}+.005s.$$

The second equation is the one generally used. If at point of maximum shearing unit stress, s becomes greater than depth, stiffeners may be omitted* except at points of concentrated loading. Here they should be figured as two angled columns, of a length equal to distance between outstanding legs, to carry entire load without assistance from web or fillers. If the equation gives values less than the depth at any point, stiffeners should be continued throughout at intervals not exceeding depth.†

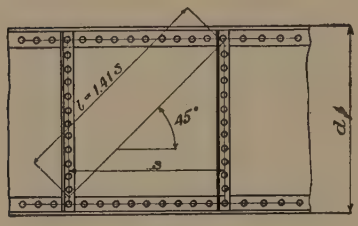


FIG. 86*a*.
Diagonal Compression in Web.

Some engineers consider that a plate girder is a Pratt truss with stiffeners for posts and the webs for diagonals. While this looks very reasonable, the weight of the evidence seems to be that the intermediate stiffeners carry little or no load.

REFERENCES

Journal Western Society of Engineers, Vol. 12, p. 788.
Engineering News, Vol. 39, pp. 322, 370.
Engineering News, Vol. 40, pp. 10, 89, 154, 186, 399.
Engineering News, Vol. 41, p. 106.

Prob. 86. A plate girder has a web $60'' \times \frac{3}{8}''$; 4 Ls, $6'' \times 6'' \times \frac{5}{8}''$; and 6 Pls $14'' \times \frac{5}{8}''$. Maximum shear 125,000 lbs.; maximum moment, 1,500,000 ft.lbs. Find spacing of stiffeners when allowable stress is 12,000 lbs. per sq.in. in flexure, tension, or pure compression.

*Because $1.41d$ equals distance between flange angles, then becomes unsupported length.

†This is apparently without theoretic foundation.

Art. 87. Computations for Beams of Variable Depth

Sudden changes in section are unadvisable, particularly where the shearing unit stress is high. Let Fig. 87*a* represent a beam in which the section *no* has a shear V, a moment of inertia I, and a width b. From Merriman's Mechanics of Materials, Art. 108, we may say that total stress in area $mnn'm'$ equals,

$$\frac{V}{I}bt\left(s+\frac{t}{2}\right).$$

Now imagine the distance mn to become infinitely small, the

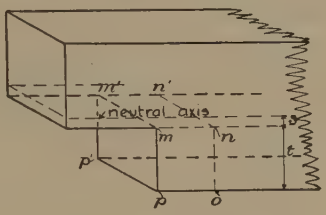


FIG. 87*a*.—Notched Beam.

above total stress remains practically unchanged as the area diminishes, hence unit stress at the point m becomes infinitely great whatever the shear. But a construction of this sort, while undesirable, will stand a limited load. This is because when elastic limit in shear is exceeded, the fibers give and release corner towards p from its share of the flexural stress. However, designs of this nature violate the fundamental principle that actual stress must not exceed elastic limit. Even the change of section when a cover plate is added is objectionable, but there is no practical way to avoid this.

When alterations are gradual, we may consider shear at any section, after deducting (or sometimes adding) vertical component of flange stress, as uniformly distributed over the area of the web. Required flange area may be determined by the exact or approximate method, using as h the depth at the point where the moment is computed. We find, however, a change in manner of computing rivet spacing.

Approximate Method for Determining Rivet Spacing

Let us proceed as in Art. 54.

Horizontal component of stress in flanges, Fig. 87*b*.

M/h at left section. $\frac{M+V\,\delta l}{h+\delta h}$ at right.

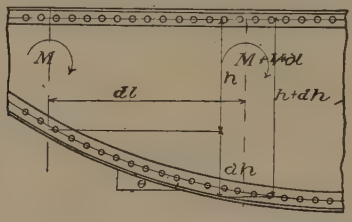


Fig. 87*b*.—Section of a Beam of Variable Depth.

Difference equals stress carried by upper rivets in length δl,

$$=\frac{M+V\,\delta l}{h+\delta h}-\frac{M}{h}$$

$$=\frac{M}{h}\left(1-\frac{\delta h}{h}\right)+\frac{V\,\delta l}{h}\left(1-\frac{\delta h}{h}\right)-\frac{M}{h} \text{ approximately}$$

$$=\frac{V\,\delta l}{h}-\frac{M\,\delta h}{h^2} \text{ approximately.}$$

Dividing by δl, we obtain horizontal shear per lineal unit,

$$=\frac{V}{h}-\frac{M}{h^2}\tan\theta.$$

Intermediate Method, see Art. 54

Letting S equal allowable fiber stress, a is

$$\frac{M}{Sh}-\frac{th}{8}.$$

$$\text{Hence flange stress at left} = aS = \frac{M}{h} - \frac{Sth}{8}.$$

$$\text{Flange stress at right} = \frac{M + V\delta l}{h + \delta h} - \frac{St(h + \delta h)}{8}.$$

Proceeding as before, the stress per lineal unit is,

$$\frac{V}{h} - \left(\frac{M}{h^2} + \frac{St}{8}\right) \tan \theta.$$

Note that the formulæ give values of horizontal components of horizontal shear. Care should be taken with signs of V and θ; the latter is plus in second and fourth quadrants for lower flanges, minus in first and third. The reverse is true for the upper flange. If both are curved, add tangents of θ algebraically with signs obtained as above. For V and M, we should use those values which, occurring simultaneously, give maximum horizontal shear.

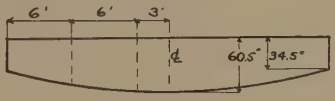


FIG. 87c.

Let us design beam of Art. 54 to be of variable depth. Span 30 feet; load per lin.ft.: dead, 1000#; live, 5000#; allowable stresses in pounds per sq.in.: shear on rivets and gross section of webs, 7200; flexural stress, 10,800; bearing 14,400. Maximum shear, 90,000#; maximum moment, 8,100,000 in.lbs. With a variable depth, economical value at center is somewhat greater than for parallel flanges; we will make it $60\frac{1}{2}''$ b. to b. angles; at ends, $90{,}000/\frac{3}{8} \cdot 7200 = 33.3$, say $34.5''$ b. to b. L's. Use $\frac{3}{8}''$ webs throughout.

APPROXIMATE METHOD

Net flange area required at center,

$$8{,}100{,}000/10{,}800 \times 57.0 = 13.15 \text{ sq.in.}$$

Use 2 L's, $6'' \times 6'' \times \frac{11}{16}''$ with a gross area of 15.56 and a net of 14.19.

Make curve of lower flange a parabola, Fig. 87*c*, and compute values of h and tan θ, with origin at center of lower flange, everything in inches. Equation of curve, $y = .0008x^2$; $\delta y/\delta x = \tan\theta = .0016x$.

Distance from center, x	0	36	108	180 inches
Ordinate, y	0	1.0	9.4	26.0 "
C. to c. gravity, h	57.0	56.0	47.6	31.0 "
Tan θ	0.0	.058	.173	.288
Maximum shear, V	+18,750 −18,750	+30,000 − 9,000	+57,000	+90,000 lbs.
Corresponding moment, M	4,725 4,725	5,184 3,888	4,320	000 kip.in.
$M \tan\theta/h$	000	5,400	15,700	000 lbs.
Remaining shear in web	18,750	24,600	31,300	90,000 lbs.
Gross shearing area	22.5	22.1	19.0	12.7 sq.in.
Shearing unit stress	830	1,120	1,650	7080 lbs. per sq.in.
Horizontal shear per lin. inch	330 330	440 230	850	2900

Intermediate Method

To obtain net area of flange required at center, we deduct $\frac{1}{8}$ gross area web from area required by approximate method, $13.15 - \frac{1}{8}\cdot\frac{3}{8}\cdot 60 = 10.3$ sq.in. Use 2 Ls $6'' \times 6'' \times \frac{1}{2}''$, gross 11.50, net, 10.50 sq.in. We will not repeat the shearing stresses, which will be slightly increased by lessened flange. We now have:

Horizontal shear per lin.in.	336 330	410 260	760	2750

In either method, these horizontal shears must be combined with vertical to obtain resultant for computing rivet pitch as in Art. 54.

Girders of variable depth save some material but at an increased cost for shop work. There is likely to be considerable bending, skew cuts, and less duplication of parts. These objections are so troublesome that girder should seldom be made in this form for economical reasons. About the only common

exception is the floor beam bracket. Other considerations may require it

(*a*) For artistic reasons. (Art. 75.)

(*b*) To save weight in operation as in cranes or drawbridges, Vol. III.

(*c*) To economize in space.

Prob. 87. Vertical beams of uniform strength spaced 5 feet apart and 20 feet between supports retain water up to top of beam. Design for allowable stresses of Art. 92. Make a general drawing of the same showing sizes of material and arrangement of rivets.

Art. 88. Loads and Specifications for Railroad Bridges

Customary provision for wind, tractive, and centrifugal forces vary little. Art. 50 well represents present practice. The locomotive loading as there specified is also typical. Sometimes provision for the trucks of heavy freight cars is required. For example, Cooper gives an alternative loading of two axles 6 feet apart, amounts of each per rail being 25,000# up to and including E 40 and 30,000 above. Also a uniform load may be employed with two, one, or no concentrations, the whole being so arranged as to produce an effect equivalent to the heaviest locomotive that may be expected to use the bridge during its life.

Impact

Loads may be subdivided as to their application.

(*a*) *Gradually applied* and *gradually removed.* Such are own weight and a crowd of people.

(*b*) *Suddenly applied.* There are all gradations, varying in time of application. A case where the interval is short is when a load is picked up quickly by a crane.

(*c*) *Applied with impact,* that is, with velocity in the direction of the force. An example is a weight dropped upon a bridge.

If we give to (*a*) the value 1.0, then (*b*) will range from 1.0 to 2.0, varying with the suddenness of the load, and (*c*) will vary from 2.0 up, increasing with the square of the velocity and decreasing with the distance in which it is brought to rest.

A perfectly adjusted locomotive on a smooth track would belong in class (*b*). Suddenness varies with different portions of the bridge, but there is quite an interval in all cases. Due to the unevenness of surface of rails, the engine and cars rock, making the load in part class (*c*).

We may further subdivide as to frequency:

(*1*) *Continuous*, for example, dead load.

(*2*) *Frequent*, at least twice a day.

(*3*) *Rare*, perhaps once or twice in life of structure.

(*4*) Only in some *extraordinary circumstances* as collision, derailment, earthquake.

(*1*) and (*2*) are most destructive. Provision for stresses mentioned in (*4*) is often omitted altogether. That is, we "take chances."

It is obviously unfair to provide for these different classes of loads with a single set of allowable stresses. There are four methods of varying permissible values according to conditions:

(*A*) Using different allowable stresses for dead, live, and wind loads. The author believes that it is unfair to include in one class the live loads in a railroad bridge with its attendant shock and frequent application, and those slowly applied and seldom attained as in high office buildings. Also that the common rule making the effect of a dead load equal to only one-half that of the same live load, underestimates the effect of the continual application of the former. In the Quebec bridge, 24,000 pounds per square inch was allowed for dead load stresses.

(*B*) Varying allowable stresses with the part of the structure. Those which are nearest the load receive it suddenly and since deflection is the least for them, velocity has there its greatest effect. We give then the lowest allowable stresses to stringers, next to floor beams and hangers, then to trusses and girders, and finally highest values for wind bracing. The latter is on account of the infrequency of the maximum loads.

(*C*) Increasing the live load by an allowance called *impact*. Such is the American Bridge Co's. formula:

$$\text{Impact in percentage of live load static stress } \frac{30{,}000}{L+300}$$

where L is the loaded length in feet producing that stress. This is a good method for trains, electric cars, and cranes, but we do not favor it for ordinary highway traffic or for the usual loads in mill or high office buildings.

(*D*) Varying allowable stress according to its range. Wöhler found that breaking unit stress varied as,

$$1+\frac{\text{minimum load}}{\text{maximum load of same kind}},$$

or

$$1-\tfrac{1}{2}\,\frac{\text{minimum load}}{\text{maximum load of opposite kind}}.$$

In Art. 50, we have preferred to use one-half the fraction in the first formula, as the allowable dead stress should not be made twice the live.

Own Weight

An approximate formula which will give weight is desirable for very rough estimates and to enable dead load stresses to be computed with enough accuracy so revision will not be necessary.

From Art. 71, the weight per foot of a built-up beam is $9.0\sqrt{M/S}$. The weight of two girders to carry a given moment would then be $12.7\sqrt{M/S}$ or $15.6l\sqrt{w/S}$ where w represents load per lin.ft. and l the length in feet. To allow for shoes, bracing, overrun at ends, and so forth, we will give,

$$W = 15l\sqrt{M/S} = 18.0l^2\sqrt{w/S},$$

where

W = total weight of steel in deck open-floor single-track bridge exclusive of track, in lbs.;

w = specified live and track load per lin.ft. in lbs. for entire bridge;

M = maximum moment caused by specified live and track load on entire bridge in in.lbs.;

S = allowable stress in flexure in lbs. per sq.in.

In case of locomotive loads, use an equivalent uniform load for moment. If S varies, use average values. Too much

reliance should not be placed upon these formulæ, as a great deal depends upon the details. The use of *crimped* stiffeners may save as much as 5% of the total weight. Designs by the exact or intermediate methods will lessen by 5 to 10% the material required in approximate computations. If economical depths are not employed, weight will be larger than that given by above formula.

For half through single-track bridges in addition to above values, let

p = panel length in feet;
d = distance between girders in feet;
a = distance between stringers in feet.

Weight of stringers is, Art. 71,

$$W_1 = 2{\cdot}7\sqrt{M/2S}\cdot \quad l = 14l\sqrt{3wp^2/4S} = 12pl\sqrt{w/S}.$$

Weight of floor beams, Arts. 71 and 77,

$$W_2 = \frac{ld}{p}\,9\sqrt{\frac{M}{S}} = 22ld\sqrt{\frac{w(d-a)}{Sp}}$$

Substituting average values, d, 15 feet; p, 14; and a, 7, and adding for extras:

$$W = \text{total weight} = 18l^2\sqrt{w/S} + 200l\sqrt{w/S} + 300l\sqrt{w/S}.$$
$$= (18l^2 + 500l)\sqrt{w/S},$$

a rough value for the weight of steel in a single-track half through open-floor bridge exclusive of rails and fastenings.

In a similar manner, for a through double track bridge:

$$W = (18l^2 + 1400l)\sqrt{\frac{w}{S}}$$

For through single-track solid-floor bridges,

$$W = (18l^2 + 1000l)\sqrt{w/S},$$

where w is the equivalent load including weight of ballast and track.

Prob. 88. What is the average value of the fraction, $1+\frac{1}{2}\frac{\text{minimum}}{\text{maximum}}$, for a six-panel through Pratt truss with inclined end post and without counters, main diagonals being made to carry compression where necessary? The dead load is one-third of the live, with two-thirds of this and entire live applied at lower chord.

Art. 89. Computation of a Half Through Single-Track Open-Floor Plate Girder Bridge

Let us design a half through, single-track, open-floor plate girder bridge with a span of 84′ 0″. Bridge level and track straight. Medium steel and $\frac{7}{8}$″ rivets are best for railroad plate girders, and we shall use them. Beams are to be com-

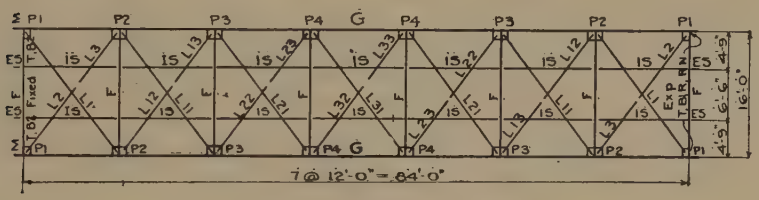


Fig. 89*a*.—Erection Diagram.

puted by the exact method. Specifications as in Art. 50, except that stiffeners are to be spaced by the rule of Art. 86, live load is to be increased by the American Bridge Co.'s impact formula, and allowable stresses made as follows, in lbs. per sq.in.:

Tension on net section	16,000
Compression on gross section	16,000 $(1-.006R)$
Shear on pins, shop rivets, and gross section webs	10,000
Shear on field rivets and bolts	7,500
Bearing on pins and shop rivets	20,000
Bearing on field rivets	15,000
Bearing on masonry	500
Bearing on rollers	$500d$

where d is the diameter of roller in inches.

With 14″ cover plates, at least 15′ 2″ will be necessary c. to c. of girders to obtain clearance. We shall make it 16′ 0″. There is a saving in odd panels; 9 would be too many, and 5 too few, as it would necessitate two I beams under each rail. We will use 7 panel lengths at 12 feet.

Design of Stringer

	Per Rail.	
	Shear. Lbs.	Moment. In Lbs.
Dead load 300 lbs. per lin.ft.	1,800	65,000
Live load, Cooper's E 50	43,800	1,200,000
Impact fraction, $300/(L+300)$	0.96	0.96
Impact	42,000	1,152,000
Totals	87,600	2,417,000

$I/c=M/S=2{,}417{,}000/16{,}000=151.0$, use one I, 24″@ 80#, $I/c=174.0$

Shearing unit stress, $\frac{87{,}600}{24\cdot 0.5}=7300$ lbs. per sq.in. O.K.

Shop rivets connecting stringer to hitch angles will be in double shear, in bearing on $\frac{1}{2}$″ web, or on two $\frac{1}{2}$″ angles. The value of the $\frac{7}{8}$″ rivets is then 8750 lbs. Use $87{,}600/8750=10$ rivets.

Design of Floor Beam (Fig. 90*b*.)

Stringers 6′ 6″ center to center. Floor beam reactions: dead, 3600#; live, 58,400#; impact fraction, 0.925; impact, 54,000#; total, 116,000#. Estimated weight of floor beam, 4000#. Maximum shear, 118,000#. Maximum moment,

$$116{,}000\cdot 57+\frac{4000}{8}\cdot 16\cdot 12=6{,}708{,}000 \text{ in.lbs.}$$

Web

118,000/10,000 = 11.8 sq.in. gross area required in shear.

Make web 10″ deeper than stringer to enable it to frame in easily with 4″×4″ flange angles in floor beam.

Required thickness = 11.80/34 = 0.35, say $\frac{3}{8}$″.

However, this would necessitate a double row of rivets in flanges and more field rivets in floor beam connection. Remember that web also aids in carrying moment. We will try $\frac{1}{2}$″.

Flanges

I/c required equals 6,708,000/16,000 = 419.

After several trials, we found that 4″×4″×$\frac{9}{16}$″ L's with 10″×$\frac{1}{2}$″ cover plates in addition to the web already decided upon, produces approximately the needed resisting moment. Shapes which will give a narrow flange have been chosen to prevent wide spaces between ties. The angles will be placed $34\frac{1}{4}$″ back to back.

	Net I in inches⁴
34″×$\frac{1}{2}$″ web = $\frac{3}{4}\cdot\frac{1}{12}\cdot\frac{1}{2}\cdot 34\cdot 34\cdot 34$	1230
4 L's, 4″×4″×$\frac{9}{16}$″ = $4\cdot 3.62(17.12-1.21)^2$	3665
2 Pls. 10″×$\frac{1}{2}$″ = $2\cdot 4.00(17.12+0.25)^2$	2420
Total	7315

$I/c = 7315/17.62 = 416.$

Cut-off Point for Cover Plate

Resisting moment (I/c) with cover plates omitted is,

$$4895/17.12 = 286.$$

Thus constituted floor beam can carry 286·16,000 equals 4,576,000 in.lbs. moment. Roughly, moment is a maximum at stringers and varies uniformly down to zero at the girder. Hence it becomes 4,576,000 in.lbs. at a point distant from the girder,

$$\frac{4{,}576{,}000}{6{,}708{,}000}57 = 38.8''$$

and theoretically plate is not required until we come to this point. Practically it should start a little nearer in order to allow two or four rivets to first take up some stress.

Gussets

Area of flange as established later, 45.75 sq.in.

$45.75 \cdot 16{,}000/10 = 73{,}200$ or 10,500# per panel (Art. 76).

Wind panel load, $250 \cdot 12 = 3000$#.

Stress in either pair of angles, using approximate method, $13{,}500 \cdot 66/19 = 47{,}000$#.

$47{,}000/16{,}000 = 2.94$ sq.in. and two L's, $4'' \times 4'' \times \frac{3}{8}''$ are ample. Stiffener angles are also strong enough.

Rivet Spacing

Between stringers where shear is practically zero, we may use the maximum spacing.

Horizontal rivets between stringer and girder:

$$s = \frac{VQ}{I} = \frac{118{,}000}{7315}(7.24 \cdot 15.91 + 4.00 \cdot 17.37) = 2960 \text{ lbs. per lin.in.}$$

Rivet value in bearing on $\frac{1}{2}''$ plate equals 8750#

Spacing equals $8750/2960 = 2.95''$.

Vertical rivets between stringer and girder:

$$s = \frac{VQ}{I} = \frac{118{,}000}{7315}(4.00 \cdot 17.37) = 1120 \text{ lbs. per lin.in.}$$

Value of two rivets in single shear is 12,000#.

Spacing is $12{,}000/1120 = 10.7''$; however, 6 is the maximum.

Connections

For erection purposes, we shall use a shelf angle under the stringer. However, the connection angles, $6'' \times 6'' \times \frac{1}{2}''$, will be computed to carry the entire stress. The field rivets which fasten these to floor beam resist either:

(1) Maximum end shear stringer in bearing on $\frac{1}{2}''$ angle.
(2) " " " " $\frac{1}{2}''$ web.
(3) " " " single shear.
(4) " floor beam reaction in bearing on two $\frac{1}{2}''$ angles.
(5) Maximum " " " $\frac{1}{2}''$ web.
(6) " " " double shear.

Obviously, (*1*) and (*2*) are alike and (*4*) has lower stresses than (*5*). Also since floor beam reaction is never smaller than end shear and never more than twice it, (*5*) has as large or larger stresses than (*1*) or (*2*), and (*3*) as large or larger than (*6*). Hence we need consider only (*3*) and (*5*).

$$(3)\quad \frac{\text{Maximum stringer shear}}{\text{Value } \frac{7}{8}'' \text{ field rivet in single shear}} = \frac{87{,}600}{4500} = 20 \text{ rivets.}$$

$$(5)\quad \frac{\text{Maximum floor beam reaction}}{\text{Value } \frac{7}{8}'' \text{ field rivet in bearing on } \frac{1}{2}''} = \frac{116{,}000}{6560} = 18 \text{ rivets.}$$

Use 20 rivets.

Connection of floor beams to double stiffeners have field rivets in double shear or in bearing on $\frac{1}{2}''$ web.

Number necessary is 118,000/6560 = 18 rivets.

Girder

Estimated weight of steel is $(500l + 18l^2)\sqrt{w/S}$, where $l = 84$, $S = 16{,}000$, and w = equivalent load for moment for a span of 84 feet plus allowance for impact plus 400 lbs. for track $= 6700\left(1 + \frac{300}{384}\right) + 400 = 12{,}300$. Substituting, we find weight of structural steel to be 148,000 lbs. or 880 lbs. per lin.ft. per girder. Total dead load is then 200 + 880 = 1080 lbs. per lin.ft. per girder.

		Shear in Kips.*				Moments in Kip Feet.		
		0–1	1–2	2–3	3–3	1	2	3
Dead		39.0	26.0	13.0	0.0	468	780	936
Loc. Loads	Maximum live	127.5	88.9	60.6	36.0	1530	2441	2870
	Wheel	(3)	(2)	(2)	(2)	(3)	(4)	(12)
	Length	77	68	56	44	77	78	80
	% Impact	79	81.5	84	87	79	79	79
	Impact	100.7	72.4	51.0	31.4	1209	1928	2270
Totals		267.2	187.3	124.6	67.4	3207	5149	6076

*1 Kip = 1000 lbs.

Note that the dead load is figured as though it were all concentrated at panel points.

Economical Depth. (Art. 71.)

$$h = 1.75\sqrt{M/S} = 1.75\sqrt{6{,}076{,}000 \cdot 12/16{,}000} = 118''.$$

We shall make it slightly less because it gives a heavier structure, better able to resist corrosion. Say $9' \cdot 0\frac{1}{2}''$ back to back of angles.

Web

Area required equals $267{,}200/10{,}000 = 26.7$ sq.in. gross. Use $108'' \times \frac{3}{8}''$ furnishing 40.5 sq.in. This is the minimum allowable thickness.

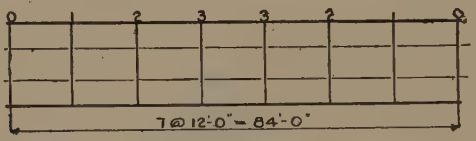


FIG. 89*b*.—Diagram of Bridge in Problem.

Stiffener Spacing

$$s = 200\left(t - \frac{V}{S_c d}\right) \quad S_c = 16{,}000,\ d = 108.$$

Section.	Shear, Lbs.	$V/S_c d$.	s, Inches.
0–1	267,200	0.155	44.0
1–2	187,300	0.108	53.4
2–3	124,600	0.072	60.6
3–3	67,400	0.039	67.2

End Stiffeners

Maximum end shear divided among two pairs of angles is 267,200 pounds. Note that reaction from end floor beam is carried down by the floor beam itself. Let us try two L's $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. They will be $1\frac{7}{8}''$ back to back, giving a radius of gyration of 3.00 ins.

Allowable total stress is $6.10 \cdot 16{,}000\left(1 - .006 \cdot \frac{108}{3.00}\right) = 76{,}500$ #.

Try two L's, $5'' \times 3\frac{1}{2}'' \times \frac{11}{16}''$.

Allowable total stress is $\frac{10.74}{6.10} \cdot 76{,}500 = 134{,}700$ lbs. O.K.

Composition of Flange

I/c required at (1) $= 3{,}207{,}000 \cdot 12/16{,}000 = 2400.$
(2) $= 5{,}149{,}000 \cdot 12/16{,}000 = 3870.$
(3) $= 6{,}076{,}000 \cdot 12/16{,}000 = 4560.$

Quantities are in inches.

Shape.	Gross.	Areas. Deduct.	Net.	Distance to c. g.
1 Web Pl. $108'' \times \frac{3}{8}''$	40.5	25%	30.4	0.0
2 Ls, $6'' \times 6'' \times \frac{3}{4}''$	16.88	4 holes 1″	13.88	52.47
1 Flge. Pl. $14'' \times \frac{1}{2}''$	7.00	2 " 1″	6.00	54.50
1 " $14'' \times \frac{1}{2}''$	7.00	2 " 1″	6.00	55.00
1 " $14'' \times \frac{1}{2}''$	7.00	2 " 1″	6.00	55.50
1 " $14'' \times \frac{9}{16}''$	7.87	2 " 1″	6.75	56.03

	For Half Section.		For Entire Section.			
	Q.	Total Q.	I.	Total I.	c.	I/c.
1 Web Pl. $108'' \times \frac{3}{8}''$			29,500			
				29,500		
2 L's. $6'' \times 6'' \times \frac{3}{4}''$	728		76,500			
		728		106,000	54.25	1950
1 Flge. Pl. $14'' \times \frac{1}{2}''$	327		35,600			
		1055		141,600	54.75	2590
1 " $14'' \times \frac{1}{2}''$	330		36,300			
		1385		177,900	55.25	3220
1 " $14'' \times \frac{1}{2}''$	333		37,000			
		1718		214,900	55.75	3850
1 " $14'' \times \frac{9}{16}''$	378		42,400			
		2096		257,300	56.31	4570

The usual rule to keep heavy plates near flange is reversed in order to provide ample splicing material.

Rivet Spacing

Horizontal rivets will be shop, in bearing on $\frac{3}{8}''$ web, value 6560 lbs.

Vertical rivets will be shop, in pairs, in single shear, value of pair, 12,000 lbs.

For Q and I, we shall use the largest values in a panel as it gives the smallest spacing.

Horizontal Rivets

Section.	Shear Lbs.	Q Ins.	I Ins.	$s = VQ/I$.	Value Lbs.	Spacing Ins.
0–1	267,200	1055	141,600	1990	6560	3.30
1–2	187,300	1718	214,900	1500	6560	4.37
2–3	124,600	2096	257,300	1010	6560	6.50
3–3	67,400	2096	257,300	550	6560	11.90

Vertical Rivets

Section.	Shear Lbs.	Q Ins.	I ins.	$s = VQ/I$.	Value Lbs.	Spacing Ins.
0–1	267,200	327	141,600	620	12,000	19.4
1–2	187,300	990	214,900	870	12,000	13.8
2–3	124,600	1368	257,300	660	12,000	18.2
3–3	67,400	1368	257,300	360	12,000	33.3

Remember, however, that $4\frac{1}{2}''$ is maximum allowable spacing.

Splices

The extreme length of $108 \times \frac{3}{8}''$ plate rolled by the Carnegie Steel Co. is $14'\ 2''$. This means that we must splice at each panel point. As we have figured the web for moment, we must splice to provide for same. Let us assume rivets to be located as shown in Fig. 90*c*. Their vertical distances from the neutral axis are then, 6, 12, 15, 18, 21, 24, 27, 30, 33, 36, 39, 42.5, 46, the remainder being ignored as they have other duties. We also omit vertical components as they are comparatively small. The allowable force on outermost rivets is 6560 lbs. Stress on any other rivet will be $6560x/46$ where x is the distance from center in inches. Its moment

will be $6560x^2/46$. The total moment carried by a half row of rivets is $6560\Sigma x^2/46 = 6560 \cdot 11{,}203/46 = 1{,}598{,}000$ in.lbs. The moment which may be carried by the gross web is $Sbh^2/6 = 16{,}000 \cdot 3 \cdot 108 \cdot 108/6 \cdot 8 = 11{,}664{,}000$ in.lbs. The moment of stress which is cut out by the hole distance x from center is $16{,}000 \cdot \frac{3}{8} \cdot 1 \cdot x^2/54 = 111x^2$ in.lbs. Total for entire section including two holes located at $x = 51.75$ in., is

$$111\Sigma x^2 = 111 \cdot 27{,}762 = 3{,}081{,}000 \text{ in.lbs.}$$

This is 26.4% deduction instead of the 25% which we had estimated. The number of rows of rivets required on each side of the joint is,

$$(11{,}664{,}000 - 3{,}081{,}000)/2 \cdot 1{,}598{,}000 = 2.69.$$

We will use 3, one of which may be field, equalling $2\frac{3}{4}$ rows.

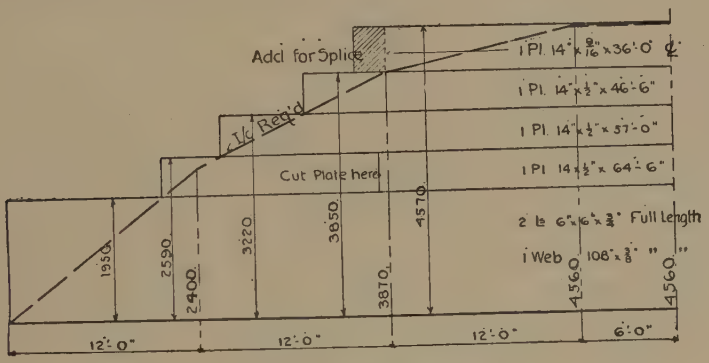


FIG. 89c.—Flange Diagram.

The minimum allowable thickness of splice plates, $\frac{3}{8}''$, obviously has a strength exceeding that of the web.

Fig. 89c shows diagrammatically the arrangement for splicing cover plate nearest flange angles. The additional length of the plate must be enough to contain rivets to fully develop strength of its net section.

$$6.00 \cdot 16{,}000/12{,}000 = 8.$$

A few extra rivets are desirable on account of the length through which they transmit stresses.

It will be difficult to arrange to drive rivets in an efficient splice for such heavy angles. Moreover, they can be obtained full length.

Bracing

The girder is 9 feet high and an ordinary freight train projects 8 feet farther; we may have 50 lbs. per sq.ft. on girder only or 30 lbs. on both; the latter gives the larger results. We shall also consider a force equal to one-thirtieth compressive strength of top flange as a short strut. Using maximum gross area, horizontal load per lin.ft. is,

$$45{\cdot}75 \cdot 16{,}000/30 \cdot 84 = 300 \text{ lbs. per lin.ft.,}$$

approximately, to be taken as live on each girder.

Total dead load, 270 lbs. per lin.ft., panel load 3.2 kips.

Total live load, 840 lbs. per lin.ft., panel load 10.0 kips.

Section	0–1	1–2	2–3	3–3
Dead shear, kips	9.6	6.4	3.2	0.0
Live shear, kips	30.0	21.4	14.3	8.6
Total shear, kips	39.6	27.8	17.5	8.6
Stress in kips (sec. $\theta = 1.25$)	49.5	34.8	22.0	10.7
Area required in sq.in. (allowable stress, 20,000 lbs. per sq.in.)	2.48	1.74	1.10	0.54
Thickness of $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times$—$L$	$\frac{1}{2}''$	$\frac{3}{8}''$	$\frac{3}{8}''$	$\frac{3}{8}''$
Net area, sq.in., deduct 1 hole 1″	2.75	2.11	2.11	2.11
Field rivets required, value 5625#	9	7	4	2

Shoes

Taking girder as just 84 feet long, we compute as follows:

Dead reaction, vertical, per rail	45,500 lbs.
Locomotive reaction, vertical, per rail	161,800 "
Impact allowance, vertical, per rail	126,300 "
Total reaction, vertical, per rail	333,600 "

Lin.in. bearing required on 6″ pin is,

$$333{,}600/20{,}000 \cdot 6 = 2.78 \text{ inches.}$$

Use two $\frac{3}{4}''$ plates on each side. Since bearings will be about

in line, shearing and bending stresses need not be computed. The design of pins will be fully taken up under " Pin Connected Truss Bridges," Chapter IX.

Maximum load on the bridge per rail = 293,700 lbs.
0.2 of the above equals horizontal force acting on line of bridge and applied at each fixed end = 58,700 lbs.
Total wind load at each corner, applied horizontally perpendicular to bridge, 1110·84/4 = 23,300 lbs.
Resultant of 58,700 and 23,300 = 63,300 lbs.
0.8·63,300/10,000 = 5.96
Use 4 bolts, $1\frac{1}{4}''$ dia.
Area required in bearing on masonry, 333,600/500 = 667 sq.in.
Linear inches of $4\frac{5}{8}''$ dia. rollers
333,600/500·4.62 = 144.

The details of the structure will be shown and explained in the following article. At this place we will compute the weights and find that they agree with those assumed. In Art. 147 will be found the computation of a box girder by the exact method. Sample computations for a through plate girder highway bridge, using approximate method, are given in Art. 93. Intermediate method is employed for deck plate girder railroad bridges for a viaduct in Art. 100.

Prob. 89. Design bridge for same data as above, except to make it of six panels and compare weights with those determined in the next article.

Art. 90. Details and Comparison of Weights

The object of *detailing* is to so design and dimension the minor parts of the structure that they have the strength demanded by the computations, at the same time making them economical to *fabricate, erect,* and *maintain.*

The actual work of detailing should begin only after design is finished for entire bridge. A common intermediate step is " making a layout," a large-scale pencil drawing of important connections. The author believes that a great deal of time

is wasted in making these for simple connections where a capable draftsman can watch clearances and so forth as he details. For example, the stringer connections need not be "laid out," but the lateral connections are best determined by this method. Let us suppose such a procedure to be followed here.

We have put 11 shop rivets in each connection angle instead of the 10 which are required, in order to give a symmetrical arrangement. The same holds true for the field rivets, but as this is an important connection, a little excess is well worth while. The field rivets in the bottom flange are for the laterals; the student should check their position. The end stringer has not been computed since it is only one-

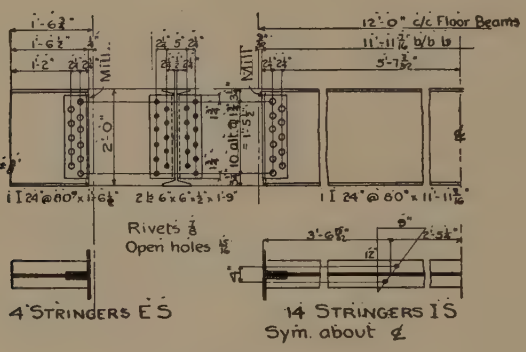


FIG. 90*a*.—Details of Stringers.

eighth as long as the intermediate, and the rails and guards do much for this short span; in case it is attempted, the rivets would probably be the weakest point.

As floor beam is symmetrical, but one-half will be given. First, we draw in center line and the girder with its stiffeners. The latter, not shown in our cut, should be in red on actual drawing. Next put in angles, web plates, and finally gussets. Then show stringer connection with shelf angles, to *match stringer*, Fig. 90*a*, bottom lateral connections to *match layout*, and connections to girder to hold computed rivets. Follow this by spacing horizontal rivets at not more than amounts obtained in the preceding article, keeping in horizontal and vertical rows to allow of multiple punching. Then make

cover plate at least six inches longer than computed value and insert vertical rivets. As they can be staggered without difficulty, it should be done. It would have been far better to have detailed the $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 8'\ 11''$ angle with the plate girder, since it must be fitted thereto. However, lack of space has prevented this here.

Details of Girder, Fig. 90*c*

We lay off flange angles and web as computed with additional length beyond center of bearing for one-half shoe, and

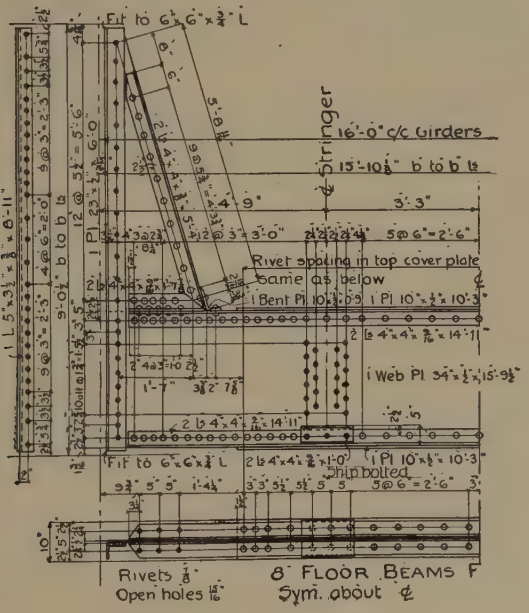


Fig. 90*b*.—Details of Floor Beam.

draw in all stiffener angles. "Breaks" should be placed where construction is regular. Next locate connections to floor beams in web and outstanding legs of stiffeners. These open holes must *match* those in *floor beams* although their effect on girders should have been considered when they were put in. Also dimension open holes for lateral plates to *match layout*. The open holes for shoe must be so placed that they will be convenient

2 GIRDERS G
Symmetrical about this Center Line
Spacing same as bottom flange
12'-0" c to c Floor Beams
Material Medium Steel
Rivets ⅞", Open Holes ⅛"
All holes punched ¾" and reamed to ⅞".
Section at Floor Beams
Other Stiffeners have no field rivets.
End View

FIG. 90c.

in both pieces which they connect. Clearance for four rivets in each shoe will be quite small. However, they may be "bucked up" from underneath all right. Then locate horizontal rivets, those in both stiffeners and flange angles being placed in gage nearest end of stiffeners. Intermediate rivets

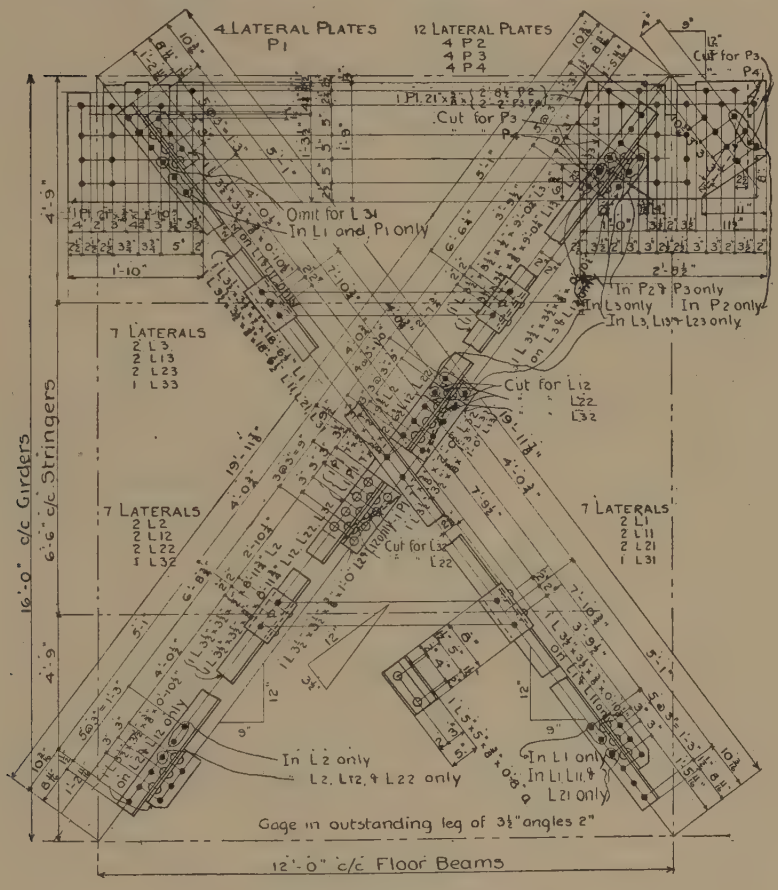

4 LATERAL PLATES P1
12 LATERAL PLATES 4 P2 4 P3 4 P4
Cut for P3
Cut for P4
Omit for L 31 In L1 and P1 only
In P2 & P3 only
In L3 only
In P2 only
In L3, L13 & L23 only
7 LATERALS 2 L3 2 L13 2 L23 1 L33
Cut for L12 L22 L32
7 LATERALS 2 L2 2 L12 2 L22 1 L32
Cut for L32 L22
7 LATERALS 2 L1 2 L11 2 L21 1 L31
16'-0" c/c Girders
6'-6" c/c Stringers
In L2 only L2, L12, & L22 only
In L1 only In L1, L11, & L21 only
Gage in outstanding leg of 3½" angles 2"
12'-0" c/c Floor Beams

FIG. 90*d*.—Details of Bracing.

in horizontal rows must be spaced so that average does not exceed computed values, especial care being taken not to hinder driving by keeping too close to stiffener. Vertical rivets may now be spaced, staggering to avoid interference in driv-

ing. For the same reason provide plenty of clearance around stiffeners. Near end of cover plates, rivets should be placed close together. Frequently a revision of spacing of horizontal rivets will be necessary. Care should be taken to keep them in horizontal and vertical rows.

Details of Bracing, Fig. 90*d*

The essential points have been determined in the layout. We will note that care must be taken to cut the plates around the stiffeners and to leave plenty of clearance between diagonals and floor beam. Since the former are on top of the plates, they will not interfere with flanges of girders but must allow room for driving of field rivets. These must be arranged and dimensioned to *match stringers*, *floor beams*, and *girders*.

Details of Shoes, Figs. 90*e*, 90*f*, 90*g*

This is a pin bearing, fixed at one end, on rollers at the other. It will be noted that there is an excess of strength in bearing area and rollers; however, the plate is not well supported at some points.

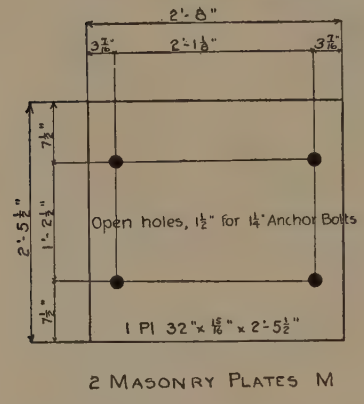


Fig. 90*e*.—Detail of Masonry Plate.

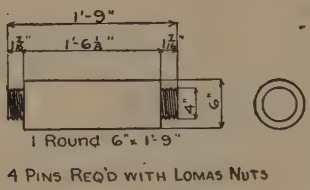


Fig. 90*f*.—Details of Pin.

The student should go over this thoroughly and see that he understands how each part connects to every other and the manner in which the expansion works. Note that we have provided for *side thrust* and *uplift* at both ends, and for expansion at free end, and for direct thrust at fixed end. Also it may be easily examined and oiled at any time.

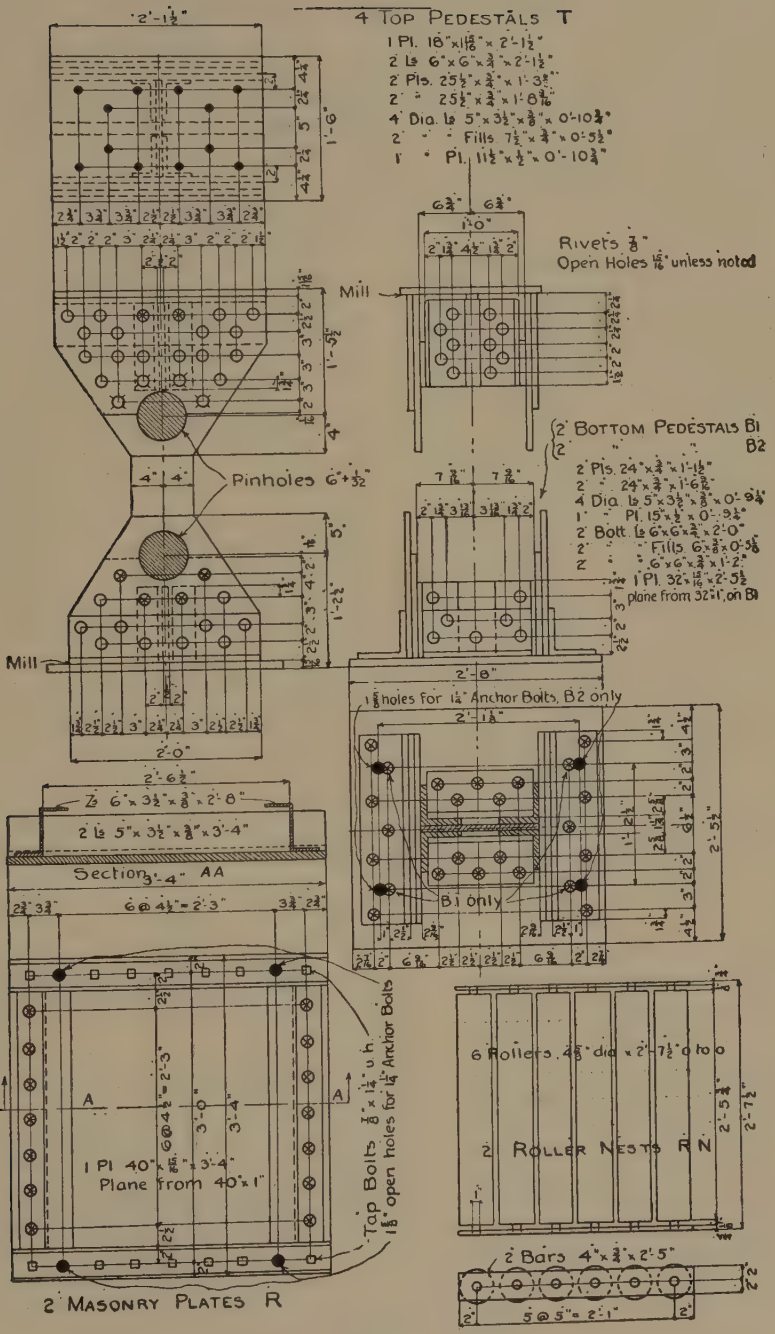


FIG. 90g.—Details of Shoes.

Comparison of Weights

A careful determination of weights without allowing anything for overrun, results as follows:

	Actual.			Estimated.	
	Lbs.	%	Lbs.	%	Lbs.
4 End Stringers	770	0.5	16,100	10.6	14,800
14 Int. Stringers	15,330	10.1			
8 Floor Beams	.....	.....	23,940	15.8	22,200
2 Girders	96,670	63.5	111,870	73.6	111,200
Shoes	10,110	6.7			
Bottom Laterals	3,780	2.5			
Field Rivets	1,310	0.9			
Totals			151,910	100.0	148,200

There is a slight excess above estimated weight, but not enough to seriously affect computations.

For estimating purposes, the following quantities are useful: (*a*) Weight of shop rivet heads compared with remainder. (*b*) Percentage which shipping weight of member is of weight of its maximum main section considered from center to center. For girders, the latter is center to center of bearing; for laterals and floor beams, center to center of girders, the former on a diagonal line.

	(*a*) Percentage Rivet Heads.	(*b*) Percentage of Main Section Center to Center.
Intermediate Stringers	1.0	115
Floor Beams	3.9	125
Girders	2.6	130
Shoes	1.8	...
Bottom Laterals	2.0	141

The weight of the web with its fittings is about 30% heavier than the flange. This is due largely to the heavy splice and filler angles. There is little doubt in this case of the economy of crimped angles or of using $\frac{3}{8}''$ splice plates with additional

fillers. We are somewhat handicapped here by a desire to make detailing simple enough to get cut down to $4'' \times 7''$. Notice also what a difference a few details can make in weight.

Camber

Bridges are sometimes curved upward such an amount

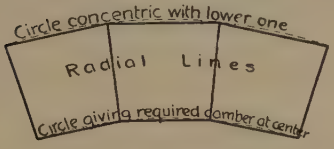


Fig. 90*h*.

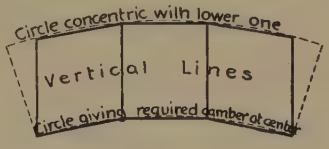


Fig. 90*i*.

Methods of Cambering Plate Girders.

that when fully loaded they will be about horizontal. This is called "cambering a bridge." The amount of the "camber" is usually about one one-thousandth of the span. Its purpose is to keep track level, to bring joints to a full bearing under the maximum load, and to preserve appearances.*

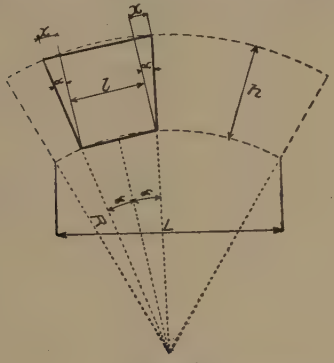


Fig. 90*j*.

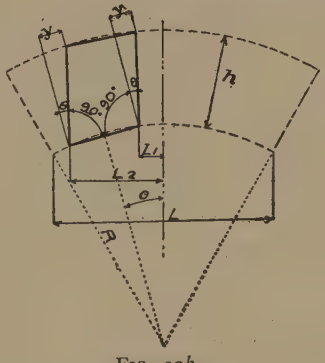


Fig. 90*k*.

Computation of Skew Cuts in Cambered Girders.

None of these amount to much for a plate girder, and we would prefer straight work and have so detailed our structure.

The method shown in Fig. 90*h* is an obvious one, and indeed a good way of cambering a truss. For a plate girder, though, it will make spacing in top and bottom flanges different.

* A false popular notion ascribes phenomenal strength to an upward curvature.

A much better method which obviates this difficulty is shown in Fig. 90*i*. This may be detailed as though it were level except that a diagram like one-half of Fig. 90*i* must be added with dimensions of skew cuts of plates. For methods of computing these cuts, see Figs. 90*j* and 90*k*.

Let c equal the cambers for girders of spans L. Required

The distance x, Fig. 90*j*,

$$x = h \tan \alpha = \frac{hl}{2R} \text{ approx.}$$

The distance y, Fig. 90*k*,

$$y = h \tan \theta = h \frac{L_1 + L_2}{2R} \text{ approx.}$$

From railroad engineering,

$$R = L^2/8c,$$

hence

$$x = \frac{4chl}{L^2} \qquad y = 4ch(L_1 + L_2)/L^2.$$

Obviously, if L_1 and L_2 are on opposite sides of the center, they should be taken with opposite signs.

Another method consists in detailing, laying out, punching and assembling as though girder were level. While still bolted, it is turned upside down on skids arranged to give a sag at center equal to required camber, and then riveted. This practically amounts to forcing girder into a camber and must result in ill-driven rivets and poor joints. This objection will be largely overcome if holes are sub punched and then reamed while thus assembled.

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E. N., Vol. 49, p. 482; Open Floors, Atchison, Topeka, and Santa Fe R. R.

E. R., Vol. 53, p. 191; Open Floors, Chicago, Burlington, and Quincy R. R.

E. R., Vol. 53, p. 74; Solid Floors, Chicago, Milwaukee, and St. Paul R. R.

Examples.* (Dimensions in Feet)

Listed in Order of Span Length

Deck Girder Bridges with Open Floor

Periodical.	Vol.	Page.	Span.	Spacing of Girders	Depth *b* to *b* Angles.	Ends.	Location.
R. R.	38	248	28.0	7.0	4.0	Square	Harriman Lines.
E. R.	52	386	65.0	6.5	6.2	.....	Davisville, R. I.
E. R.	59	155	66.5	8.5	6.5	Square	Newcastle Jc., Pa.
E. R.	64	524	80.0	....	8.5	.....	Pend Oreille, Wash.
E. R.	55	520	83.5	6.5	9.0	Skew	Warehouse Pt., N. H.
R. R.	27	35	85.0	7.0	8.7	Skew	Turtle Creek, Pa.
E. R.	52	386	100	....	9.2	.....	Shannock, R. I.
E. N.	27	316	100.5	7.5	8.0	Skew	Beaver River, Pa.
E. R.	49	752	104	6.0	9.0	Square	Monocacy.
E. R.	52	386	106	....	9.2	.....	Naugatuck Jc., Conn.
R. R.	22	352	112	8.5	9.0	Square	
E. R.	53	605	119.3	8.0	9.5	Square	Chester, Pa.
E. R.	51	530	125.3	8.0	9.5	Skew	Pine Creek.
E. R.	56	522	126.5	....	10.0	Skew	Towanda, Pa.
E. R.	52	324	128	9.0	9.5	Square	Hubbard, Ohio.
E. N.	51	166					

Through or Half Through Girder Bridges with Open Floors

E. N.	27	345	53.0	14.2	4.2	.....	C. & N. W. R. R.
E. R.	49	770	54.0	16.0	7.0	.....	Newark, N. J.
E. R.	52	407	76	15.5	9.0	Skew	Providence, R. I.
E. R.	51	68	84.5	18.8	9.5	Skew	Bethlehem, Pa.
E. R.	49	770	95.0	16.0	10.0	.....	Newark, N. J.
E. R.	49	383	100	35.0	9.7	Skew	Clairton, Pa.
E. N.	27	316	100	13.0	8.0	Square	Berlin, Conn.
E. R.	52	407	102.7	16.0	8.9	Square	Ashley Falls, Mass.
E. R.	49	770	106.5	16.0	10.0	.....	Newark, N. J.
E. R.	43	102	107.7	28.8	11.2	Square	Clyde River, N. Y.
E. N.	51	166	111	17.7	10.0	Square	Yellow Creek.
E. R.	49	491	114.0	29.0	10.0	Skew	Chicago, Ill.
E. R.	41	565	125.2	16.8	9.0	Skew	Erie R. R.

* See also "Types and Details of Bridge Construction," by Skinner. Vol. II, Plate Girders.

DECK GIRDER BRIDGES WITH SOLID FLOORS

Periodical.	Vol.	Page.	Span.	Spacing of Girders	Depth b to b Angles.	Ends.	Location.
E. R.	40	6	42.0	7.0	5.0	Square	C. and N. W. Ry.
E. R.	65	380	58.0	7.5	6.8	Square	Connellsville, Pa.
R. R.	25	499	58.0	8.2	6.0	Square	
E. R.	59	548	73.5	8.5	7.5	.....	Turtle Creek, Pa.
E. C.	40	174	76	6.0	5.0	Square	Boston, Electric R. R.
E. R.	67	355	114.3	8.5	6.5	Skew	Akron, Ohio.
E. R.	55	728	115.3	7.0	9.5	Square	Scranton, Pa.
E. R.	53	745	Various				

THROUGH OR HALF THROUGH GIRDER BRIDGES WITH SOLID FLOORS

E. R.	40	71	22.0	....	3.7	Square	C. and N. W. Ry.
E. R.	59	548	32.0	13.0	4.3	.....	Mahoningtown, Pa.
E. R.	65	265	32.3	13.0	4.5	Square	Cranford, N. J.
E. R.	65	380	48.0	....	6.5	.....	Connellsville, Pa.
E. R.	60	50	54.0	13.0	5.5	Skew	Woodside, N. Y.
E. N.	32	291	.....	13.0	4.2	Skew	Chicago, Ill.
E. R.	66	50	60.4	....	....	Skew	W. Allis, Wis.
R. R.	27	178	62.0	13.0	5.0	Square	Chicago, Ill.
E. N.	36	114	68	13.0	6.0	Square	Chicago, Ill.
E. R.	46	152	72	15.5	7.0	Skew	Lake Erie and W. R. R.
E. R.	51	498	78.3	....	6.7	Skew	Walnut Lane, Pa.
E. R.	67	355	80.0	14.5	7.5	Skew	Akron, Ohio.
E. R.	64	226	82.5	29.5	10.0	Skew	B. & O.
E. R.	42	318	85	....	10.0	.....	Oriskany, N. Y.
E. R.	40	474 498	113.5	29.0	10.0	Skew	Maiden Lane, N. Y.
E. R.	54	209	Various				Chicago, Ill.

Prob. 90. Determine number of field rivets required of each grip in above girder bridge.

Art. 91. Loads for Plate Girder Highway Bridges

Railroad specifications are well standarized and we will rarely see one which admits of inferior work. With highway bridges we find large differences due to variations in engineering. To a certain extent less stringent rules are suitable; the full load comes seldom or perhaps never, and it is gradual in its application; further, there is far less impact. But it

must be remembered that its heaviest weight is a crowd of people and steel is cheap beside the load which it carries. This load is usually assumed at a lower value than a dense crowd, and it is often reduced for long or country bridges. Yet it is possible that these may be loaded with 125 pounds per square foot from end to end by some unusual accident or spectacle, a drowning, fire, race, or aerial performance. In cities the police are present to prevent any undue loading, but in the country there would be no adequate restraining force.

The loads which act on highway bridges are:

(1) Dead, (2) Live, (3) Wind, (4) Tractive, (5) Centrifugal, (6) Snow.

(1) *Dead Load*

Weight of wood may be computed by using Art. 2; to obtain that for solid floor, see Art. 74. To determine roughly weight of steel, we will follow method of Art. 88.

Let w = weight of floor and uniform live load in lbs. per sq.ft.;
l = length center to center of bearings in feet;
p = panel length in feet;
b = breadth = center to center girders plus $\frac{1}{2}$ sidewalks in feet;
a = spacing center to center of stringers in feet;
S = allowable fiber stress in lbs. per sq.in.

From Art. 71, weight of built beam, $W = 9l\sqrt{M/S}$;
weight of rolled beam $= 7l\sqrt{M/S}$.

Now in using these we must first change M into foot pounds, then increase to allow for effect of concentrated loads as explained later and finally add for extras. See Table on next page. Then let us substitute for p and a, average values of 15 and 2.5 respectively, obtaining approximately:

$$W = bl\sqrt{\frac{w}{S}}\left(16\frac{l}{\sqrt{b}} + 4b + 150\right).$$

for the total weight of steel, except for flooring, in either deck or through plate girder bridges with steel stringers. If weight

of steel is desired for bridges with wooden stringers, omit the last term.

	Girders Built Up.	Floor Beams Built Up.	Stringers Rolled.
Weight per foot, Art. 71.....	$9\sqrt{M/S}$	$9\sqrt{M/S}$	$7\sqrt{M/S}$
Change to feet.....	$9\sqrt{\frac{12wbl^2}{8\cdot 2S}}$	$9\sqrt{\frac{12wpb^2}{8S}}$	$7\sqrt{\frac{12awp^2}{8S}}$
Simplify and multiply by length	$2l7.8\sqrt{\frac{wbl^2}{S}}$	$\frac{bl}{p}11\sqrt{\frac{wpb^2}{S}}$	$\frac{bl}{a}8.6\sqrt{\frac{awp^2}{S}}$
Simplify.....	$15.6l^2\sqrt{b}\sqrt{\frac{w}{S}}$	$11\frac{l}{\sqrt{p}}b^2\sqrt{\frac{w}{S}}$	$8.6\frac{bl}{\sqrt{a}}p\sqrt{\frac{w}{S}}$
Add for extras and concentrated loads.....	$16l^2\sqrt{b}\sqrt{\frac{w}{S}}$	$16\frac{l}{\sqrt{p}}b^2\sqrt{\frac{w}{S}}$	$15\frac{b}{\sqrt{a}}lp\sqrt{\frac{w}{S}}$

(2) *Live Loads*

These may be:

(*a*) Crowd of People.
(*b*) Electric Car.
(*c*) Road Roller.
(*d*) Automobile or Wagon.

(*a*) Tests have shown that 181 lbs. per sq.ft. of picked men may be forced into an enclosure.* But the customary allowance is 80 to 100. Students should test these values for themselves by marking off a space on the floor and standing therein persons of known weight. This experiment will make one somewhat skeptical about the sufficiency of present allowances. Nevertheless, we will, in a way, follow current practice in our specifications.

(*b*) Electric Car. The weight of these vehicles with their freight is usually specified as being equally divided between two, three, or four axles. The distance supposed to be occupied by the car and from which other load is excluded should also be mentioned. Another point which needs consideration is the interval between cars. City bridges, carrying a heavy

* American Society of Civil Engineers: Vol. 54, Part 7, p. 441.

traffic, might have a practically continuous line due to a blockade, while the operation of a country line could preclude the possibility of more than one car on the bridge at a time. There may be one or two tracks, located centrally or at one side. Rails should, if possible, be directly supported by stringers. Cooper specifies for heavy electric railways, 24 tons on two axles 10 feet apart for floor and loads per lin. ft. per track on girders varying from 1200 to 1800 lbs., decreasin with the span; for light electric railways, 18 tons on two axles 10 feet apart for floor, and loads per lin.ft. per track on girders of 1000 to 1200.

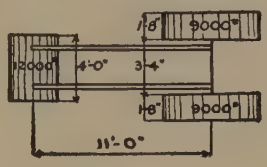

FIG. 91*a*.
Diagram of a Road Roller.

(*c*) Road Roller. Outside of car tracks and for bridges where there is no probability of electric cars, the road roller will often be the heaviest load. Although sometimes heavier, 30,000 lbs. is frequently specified as seen in Fig. 91*a*. Only one is supposed to be on a span at a time and usually the trusses are not figured for them.

(*d*) Automobile or Wagon. Where there are neither cars nor road rollers, heavy wagons or motor trucks try the floor system most severely. These will vary a good deal with the location. Waddell specifies for light country service, 10,000 lbs. total on two axles 8 feet apart with wheels 6 feet center to center transversely. Like road rollers, only one is supposed to be on the span and trusses are not figured for them.

These loads are rather complicated and a simplified loading will be given in our specifications. Nevertheless, this conventional system should be carefully compared with existing and probable loads before they are employed.

(*3*) *Wind Loads*

The same wind loads act as on a railroad bridge. However, as they rise in value, the live load is correspondingly reduced, pedestrians, teams, and finally electric cars seeking shelter. Hence wind load need not be considered in combination with live. We might follow some authors in using full value of the wind pressure, some 50 lbs. per sq.ft., and

allow higher stresses on account of infrequency. It is preferable, however, to design for a moderate value, 40 lbs. per sq.ft., and employ same allowable unit stresses. Obviously, the two methods produce about the same results. This pressure should also be considered as applied vertically upwards to the bottom. If this force is more than the weight, sufficient steel must be provided to anchor securely. This latter clause is intended to prevent accidents such as occurred at Minneapolis on August 20, 1904.*

(4) *Tractive Loads*

These forces are small in amount and need not be computed. In design, however, stiffness should be given in a longitudinal direction, the amount varying with the traffic.

(5) *Centrifugal Loads*

Speed is usually low; also, as already noted, highway bridges are seldom built on a curve.

(6) *Snow Loads*

The conditions which would allow the accumulation of an appreciable amount of snow would lessen by an equal amount the probable live load. It is perhaps best to make some allowance therefor in Canadian bridges.

Prob. 91. Find approximate weight of steel in a highway plate girder 100 feet long with plank flooring. It is 30 feet between girders and has one 8-foot sidewalk. Loading 100 lbs. per sq.ft.

* Eng. News, Vol. 52, p. 192.

Art. 92. Specifications for Plate Girder Highway Bridges

We will use Art. 50 except as noted below.

(*a*) *Description.* (Replacing (*a*) of Art. 50)

(*1*) PREFERRED TYPES

Spans up to 35 feet. Rolled I-beam bridges.
35 to 80 feet. Plate girders.
80 to 150 feet. Riveted Pratt or Warren trusses.
150 to 200 feet. Pin-connected Bowstring trusses.
200 to 600 feet. Pin-connected Pennsylvania trusses.

(*2*) PREFERRED DEPTH

Depth should not be less than the following amounts:
I-beam bridges. $\frac{1}{30}$ span.
Plate girders, $\frac{1}{12}$ span.
Truss bridges. 10 feet plus $\frac{1}{15}$ span.

(*3*) SPACING OF GIRDERS AND TRUSSES

Center to center spacing of girders and trusses shall not be less than one-fifteenth span.

(*4*) CLEARANCE DIAGRAMS

All parts of through trusses must give clearances not less than diagrams below:

(*5*) FENCE

An ornamental fence not less than 3′ 6″ high is to be provided. It shall be figured (*a*) for a load per lineal foot equal to the uniform load per sq. ft., applied horizontally and acting outwards; or (*b*), the assumed wind pressure acting horizontally in either direction.

(6) PLANK FLOORS

Single-layer plank floors for roadway shall be not less than $2\frac{3}{4}''$ thick, of oak or yellow pine, nailed to each stringer with $\frac{1}{4}''$ joints. For two layers, the lower shall be as above, the upper of $1\frac{3}{8}''$ spruce or white oak, laid tight. Footwalk plank shall be not less than $1\frac{3}{4}''$ thick, laid at right angles to the bridge with $\frac{1}{4}''$ openings. Concentrated load shall be considered as distributed over 24″ in width for one layer and 36″ for two layers.

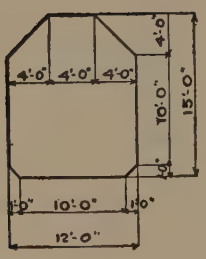

FIG. 92*a*.—Single-track Electric Car. Width center to center tracks, 10 feet.

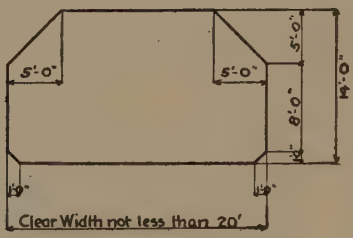

FIG. 92*b*.—Highway.

(7) SOLID FLOOR

	Roadway.	Sidewalk
Minimum thickness of plate	$\frac{5}{16}''$	$\frac{1}{4}''$
Minimum buckle for buckled plate	2	2
Minimum concrete above plate	3	2

(8) WOODEN STRINGERS

Oak, yellow pine, and Oregon pine are preferred. Rectangular beams, firmly fastened at ends and bridged every 10 feet may have a depth of 7 times width. Otherwise depth should not exceed 4 times width.

(9) STEEL STRINGERS

Steel stringers shall have a depth not less than one-twentieth their span for electric cars, one-twenty-fifth for rest of roadway, and one-thirtieth for sidewalks.

Maximum allowable spacing c. to c.

(10) STRINGER SPACING

Plank floor, wooden joists, roadway	2′ 0″
Plank floor, wooden joists, sidewalk	3 0
Plank floor, steel joists, roadway	3 0
Plank floor, steel joists, sidewalk	4 0
Solid floor	5 0

(*b*) *Loads.* (Replacing (*b*) of Art. 50)

(1) DEAD LOADS

Dead loads shall be computed in all cases. Timber to be considered as green and estimated as given in Art. 2. Assume solid floors to weigh amounts stated in Art. 74.

(2) OWN WEIGHT

Stresses due to own weight must be allowed for in the design.

Loading 100 shall be as follows:

100 lbs. per sq.ft. on all floors including sidewalks.

(3) LOADING 100

In addition to this, on any part of the roadway, 8 axle loads spaced 6–16–6–12–6–16–6 feet apart, each axle being 10,000 lbs., distributed between two wheels 5 feet apart. Above cars must always be parallel to the bridge.

Other loadings are proportional in amount. For example, loading 80 is 80 lbs. per sq.ft. and has axle loads of 8000 lbs.

(4) PROPER LOADINGS

For city bridges carrying heavy traffic use	Loading 120
For ordinary city and suburban bridges	Loading 100
For country bridges	Loading 80
For light country bridges	Loading 70

(5) WIND LOADS

The pressure of the wind shall be taken as 40 lbs. per sq.ft. on the unloaded structure, acting horizontally on one girder only. Bridge shall also be designed for same unit force acting upward.

(6) TRACTIVE AND CENTRIFUGAL FORCES

If tractive and centrifugal forces of a considerable amount occur, they may be allowed for by methods similar to those followed in Art. 50.

(c) *Allowable Stresses.* (Replacing (c) of Art. 50)

Let us take D to represent the safe tensile strength of material equals 13,500 for soft steel, 15,000 for medium steel, and 22,500 for nickel steel, all in pounds per sq.in. Also let R be the maximum slenderness ratio.

(1) ALLOWABLE UNIT STRESSES

Tension on net section	D
Compression on gross section	$D(1-.006R)$
Shear on shop rivets, pins, and gross section of webs	$2D/3$
Shear on field rivets and bolts	$0.5D$
Bearing on shop rivets and pins	$4D/3$
Bearing on field rivets and bolts	D
Bending on pins	$1.5\ D$
Other flexural stresses	D

(2) OTHER ALLOWABLE STRESSES

For wood, see Art. 6. Allowable stress per lineal inch on medium steel rollers is 400 times diameter in inches. For bearing on masonry, 400 pounds per sq. in.

(Change (*d* 2) to read as follows:) Except for fillers, lacing bars, and buckled plates for sidewalks, minimum thickness of metal is $\frac{5}{16}''$.

In (*d* 9), maximum allowable slenderness ratios for wind and other struts should be made 150 and 125 respectively.

Eliminate last sentence of (*f* 2).

Where electric cars are the sole traffic or a large part of it, specifications should be made more like railroad bridges.

REFERENCES

Cooper's General Specifications for Steel Highway and Electric Railway Bridges and Viaducts.

Engineering Record, Vol. 68, p. 230.

Prob. 92. What uniform load would be equivalent to loading 100 for the girders of a bridge with a 40-foot roadway and two 8-foot sidewalks?

Art. 93. Computation of a Plate Girder Highway Bridge

Let it be required to design a city bridge of 60 feet span for a heavy traffic. It is to have a roadway 30 feet in the clear and a sidewalk on either side 7 feet net width. We will use plank floors, steel joists, and half through girders. Loading 120, $\frac{3}{4}''$ rivets, material medium steel, except rivets of soft steel. Use specifications of preceding article except that built-up beams are to be computed by the approximate method and stiffeners will be spaced by the theoretic formula.

Divide into 5 panels of 12 feet each with reaction stringers. Section as shown in Fig. 93*a*.

Roadway Plank

Span, 3′ 0″. Let us use a $1\frac{3}{8}''$ wearing surface of spruce and a lower layer of white oak.

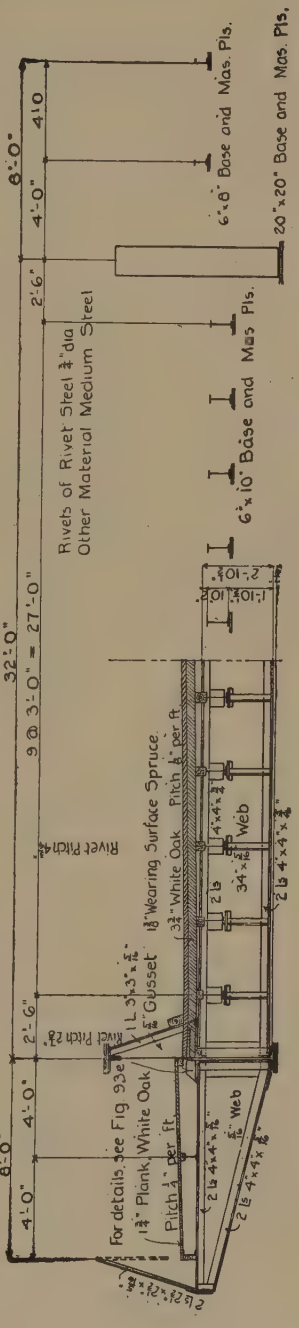

Fig. 93*a*.—Half Intermediate Section, Half End View.

Uniform load, 120+20 (est.) = 140 lbs. per sq.ft.
Concentrated load, 6000/3 = 2000 lbs. per lin.ft.
Maximum shear, 210+2000 = 2210 lbs.
Maximum moment, 1890+18,000 = 20,000 in.lbs.
} on strip 1 foot wide.

Required depth in shear is

$$3V/2bS_s = 3 \cdot 2210/2 \cdot 12 \cdot 100 = 2.76''.$$

Required depth in moment is

$$\sqrt{6M/bS} = \sqrt{6 \cdot 20000/12 \cdot 750} = 3.65''.$$

Use $3\frac{3}{4}''$ white oak plank.

Sidewalk Plank

Span, 4′ 0″, material, white oak.
Uniform load, 120+10 (est.) = 130 lbs. per sq.ft.
Maximum shear, 260 lbs.
Maximum moment, 3120 in.lbs.
} on a strip 1 foot wide.

Required depth in shear is

$$3V/2bS_s = 3 \cdot 260/2 \cdot 12 \cdot 100 = 0.33''.$$

Required depth in moment is

$$\sqrt{6M/bS} = \sqrt{6 \cdot 3120/12 \cdot 750} = 1.44''.$$

Use $1\frac{3}{4}''$ white oak plank.

Roadway Joists. Fig. 93*b*.

3′ 0″ on centers. 12′ 0″ long.
Uniform load per lin.ft., 40 (est.)+3 · 140 = 460 lbs.
Concentrated loads, two, 6 feet apart, 6000 lbs. each.
Maximum shear is 460 · 6+6000+3000 = 11,800 lbs.
Maximum moment is 99,400+243,000 = 342,000 in.lbs.
Required $I/c = M/S = 342{,}000/15{,}000 = 22.8$.
Use one I, 10″ @25#. $I/c = 24.4$.
Shearing unit stress is 11,800/10 · 0.31 = 3800 lbs. per sq.in. O. K.

Sidewalk Joists

4′ 0″ on centers. 12′ 0″ long.
Uniform load per lin.ft., 20 (est.) + 130·4 = 540 lbs.
Maximum shear is 3240 lbs.
Maximum moment, 116,600 in.lbs.
Required I/c is 116,600/15,000 = 7.8.
Use one I, 6″@14.75#, I/c = 8.0.
Shearing unit stress, 3240/6·0.35 = 1550 lbs. per sq.in. O.K.
The fascia stringer will have but half the load and a 6″ ⊏ @10.5# is obviously strong enough.

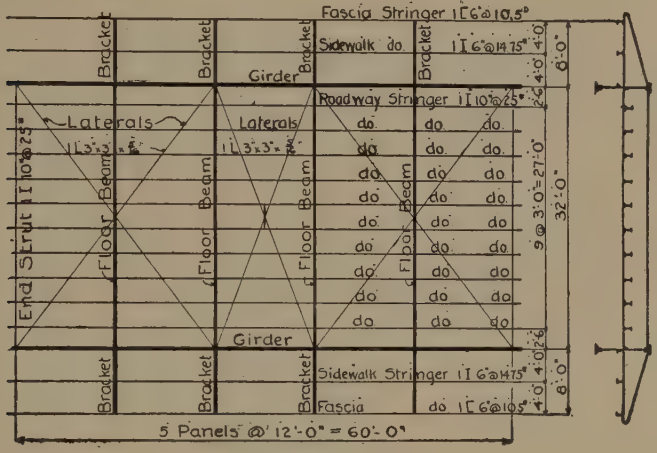
Fascia Stringer 1 C 6 @ 10.5#
Sidewalk do. 1 I 6 @ 14.75#
Bracket
Girder
Roadway Stringer 1 I 10 @ 25#
Laterals
1 L 3½ × 3½ × ⅝
End Strut 1 I 10 @ 25#
Floor Beams
do.
Sidewalk Stringer 1 I 6 @ 14.75#
Fascia do. 1 C 6 @ 10.5#
4′-0″
6′-0″
9 @ 3′-0″ = 27′-0″
32′-0″
5 Panels @ 12′-0″ = 60′-0″

Fig. 93*b*.

Floor Beams (Fig. 93*c*).

Dead weight sidewalk, 80 (est.) + 50 + 120 = 250 lbs. per lin.ft.
bracket joists plank

Dead weight roadway, 180 (est.) + 120 + 300 = 600 lbs. per lin.ft.
plank joists floor beam

Live load 1440 lbs. per lin.ft. on entire span plus concentrations on roadway only. Maximum floor beam reactions due to latter are two of 9000 lbs., 5 feet apart. In computing shears and moments, we may ignore stringer action without sensible error.

Values are in feet and pounds.

Lead.	Shear in Kips.				Moment in Kip Feet.		
	Outside of Girder.	Inside of Girder.	8 Feet from Girder.	Center.	Girder.	8 Feet from Girder.	16 Feet from Girder.
Dead.....	2.0	9.6	4.8	0.0	8.0	49.6	68.8
Uniform Live..	11.5	23.0	13.0	5.8	46.0	138.2	184.3
Concen. Live...	....	16.6	12.0	7.6	....	96.0	121.6
Maximum.....	13.5	49.2	29.8	13.4	54.0	283.8	374.7

$$h = 1.75\sqrt{M/S} = 1.75\sqrt{374{,}700 \cdot 12/15{,}000} = 30.4''.$$

However, the fact that there will be no stiffeners and no reduction in the size of flanges. tends to increase economical depth. We will make it 34″

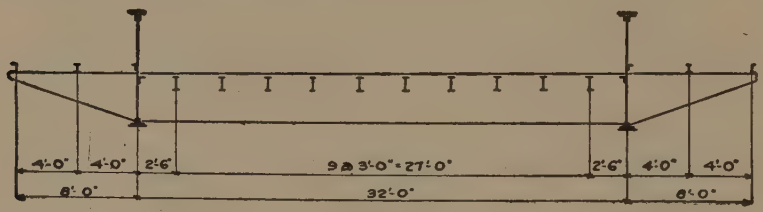

FIG. 93c.

Thickness required equals $49{,}200/10{,}000 \cdot 34 = .145''$. Make $\frac{5}{16}''$.

Spacing stiffeners, $s = 200\left(t - \frac{V}{S_c d}\right) = 43''$, none necessary.

Net area required in flange is $\frac{374{,}700 \cdot 12}{15{,}000 \cdot 32.5} = 9.22$ sq.in.

Use two Ls, $4'' \times 4'' \times \frac{3}{4}''$, gross area 10.88, net 9.57 sq.in. For brackets, $4'' \times 4'' \times \frac{5}{16}''$ will evidently suffice.

Rivet spacing $= Rh/V$, where R represents rivet value, in this case, 4220 lbs.

Spacing at inside of girder, $4220 \cdot 32.5/49{,}200 = 2.79''$.

Spacing 8′ from girder, $4220 \cdot 32.5/29{,}800 = 4.60''$.

At outside of girder and center the maximum of 6″.

The shop rivets connecting angles to roadway stringers will be in bearing on web rather than double shear.

$$\frac{11{,}800}{18{,}000 \cdot \frac{3}{4} \cdot 0.31} = 3 \text{ rivets.}$$

The field rivets connecting angles to floor beams will be in single shear carrying maximum end shear stringer.

$$\frac{11{,}800}{6750 \cdot 0.44} = 4 \text{ rivets,}$$

or in bearing on $\frac{5}{16}''$ web carrying maximum floor beam reaction.

$$\frac{14{,}600}{13{,}500 \cdot \frac{3}{4} \cdot 0.31} = 5 \text{ rivets; use this.}$$

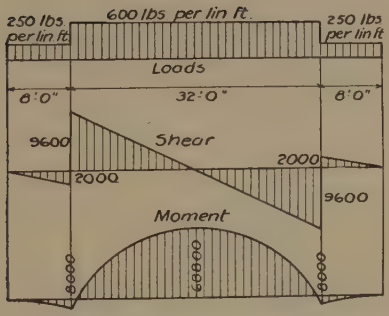

FIG. 93*d*.—Shear and Moment Diagram for Dead Loads on Floor Beam.

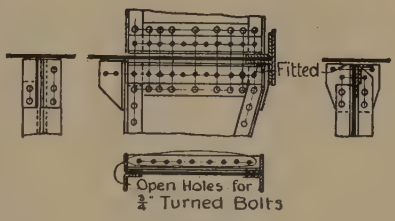

FIG. 93*e*.

The shelf angles are often computed to carry entire load. However, the above is about the minimum which should be placed in the angles anyway.

The field rivets connecting floor beam to girder will be in single shear and their value is 2970 lbs.

$$49{,}200/2970 = 17 \text{ rivets.}$$

The field rivets connecting sidewalk bracket to girder will carry both shear and moment. A common error is to consider the former only; on the other hand, the latter is the more important, so much so that it is usually the only one that

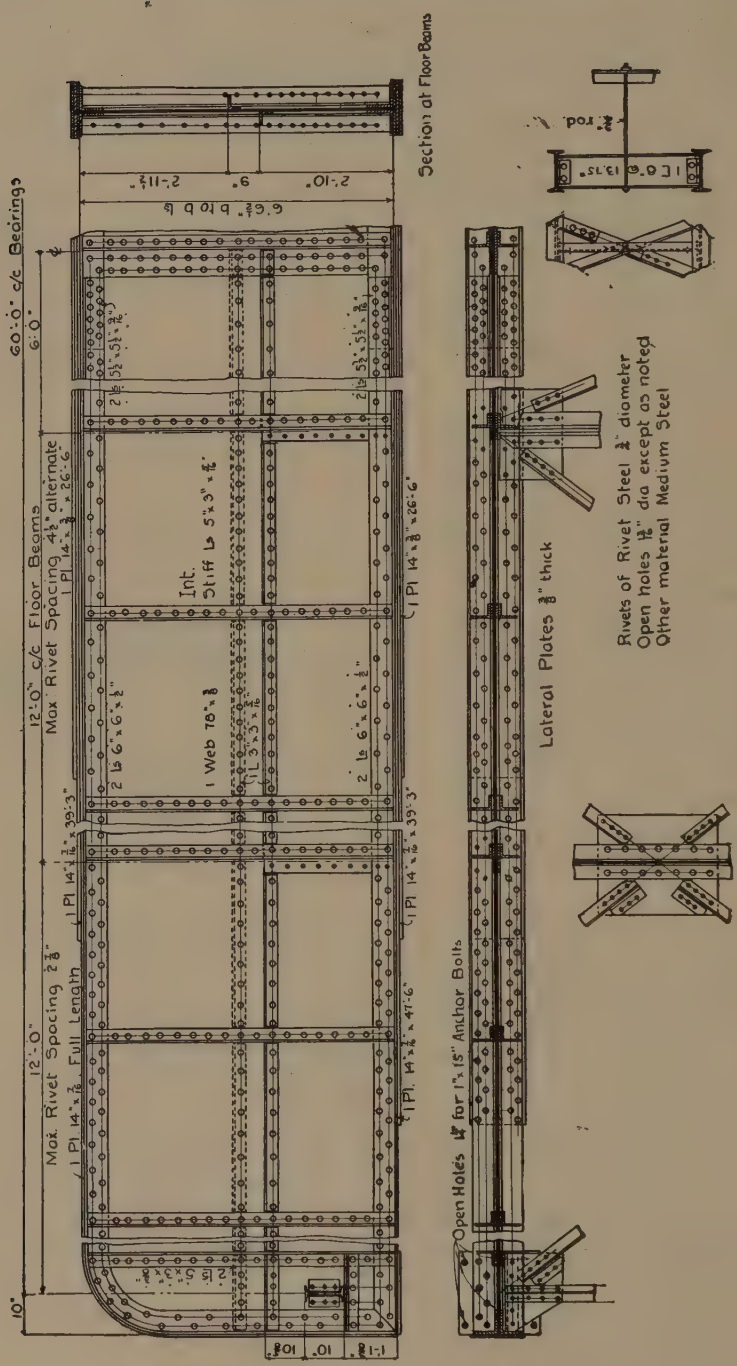
Section at Floor Beams
60'-0" c/c Bearings
12'-0" c/c Floor Beams
Max. Rivet Spacing 4½" alternate
Max. Rivet Spacing 2⅞"
Full Length
Int. Stiff Ls 5"x3"x7/16"
1 Web 78"x⅜
Lateral Plates ⅜" thick
Open Holes 1¼" for 1"x15" Anchor Bolts
Rivets of Rivet Steel ¾" diameter
Open holes 13/16" dia except as noted
Other material Medium Steel

FIG. 93f.

need be considered. Let us take the outer rivets as 30″ apart with intermediate 3″ spacing, Fig. 93*e*. Further, let us have the lower angle fitted against girder thus enabling it to carry compression due to moment. Then assuming lower rivet as a fulcrum, $\Sigma x^2 = 3465$ and

$$M = R\Sigma x^2/30 = 2970 \cdot 3465/30 = 343{,}000 \text{ in.lbs.},$$

or more than one-half of what is required. We might slot the web of the girder but this weakens it, and is expensive in the shop. Larger rivets might be employed but this is objectionable for reasons discussed in Art. 47, (*4*). So we will proceed as follows: Put on additional angle, thus bringing rivets into double shear. To avoid entering joints, it is field riveted to both girder and bracket. Fillers will be fastened to web of floor beam by an extra row of rivets, thus making bearing value greater than shear. As a further precaution, horizontal legs of bracket angles are fastened to girder by a bent plate at the bottom and by an angle at top. Amount to be provided for is,

$$54{,}000 \cdot 12 = 648{,}000 \text{ in.lbs.}$$

$$M = \frac{R\Sigma x^2}{30} = \frac{5940 \cdot 3465}{30} = 686{,}000 \text{ in.lbs.}$$

GIRDER

Dead concentrations at floor beam,

$$\underbrace{25 \cdot 16 \cdot 12 + 10 \cdot 8 \cdot 12}_{\text{Flooring}} + \underbrace{150 \cdot 12 + 30 \cdot 12}_{\text{Stringers}} + \underbrace{140 \cdot 24}_{\text{Floor Beams}} = 11{,}280;$$

say, 12,000#.

Estimated dead load of each girder is,

$$8l^2\sqrt{\frac{w}{S}b} = 8 \cdot 3600\sqrt{\frac{145 \cdot 40}{15{,}000}} = 18{,}000\#,$$

or 300# per lin.ft.

For drawing of girder see Fig. 93*f*.

Heaviest stress due to uniform live load will occur when

roadway and near sidewalk is loaded and far sidewalk empty. Panel concentrations then are,

$$120 \cdot 12 \cdot 40 \cdot 20/32 = 36{,}000\,\#$$ for uniform load.

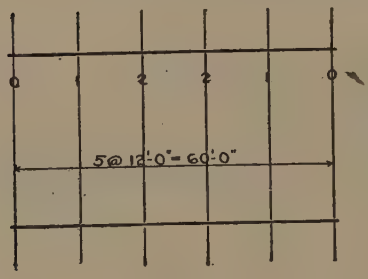

FIG. 93g.

For the axle loads we shall assume one wheel over girder although it probably can be placed no nearer than one foot. The axle concentrations then become

$$6000 + 6000 \cdot \frac{27}{32} = 11{,}000\,\#$$

spaced 6–16–6–12–6–16–6 feet. To get maximum shears and moments, follow rules for locomotive loadings.

Loads.	Shears in Kips.			Moments in Kip Inches.	
	0–1	1–2	2–2	1	2
Dead, girder.....	9.0	5.4	1.8	1,037	1,620 *
Dead, floor.....	24.0	12.0	....	3,456	5,184
Live, uniform.....	72.0	43.2	21.6	10,368	15,552
Live, concentrated...	29.3	16.1	8.1	4,224	6,072
Total.....	134.3	76.7	31.5	19,085	28,428

* Computed at center.

Economical depth is $h = 1.75\sqrt{\frac{M}{S}} = 1.75\sqrt{\frac{28{,}428{,}000}{15{,}000}} = 76''$, say $78''$.

Thickness of web is $\frac{134{,}300}{78 \cdot 10{,}000} = .172''$; although $\frac{5}{16}''$ is allowable, we will employ $\frac{3}{8}''$ as it lessens rivets and stiffeners and will longer resist corrosion.

Stiffeners

Intermediate stiffeners will be $5'' \times 3'' \times \frac{5}{16}''$, longer leg outstanding. When $1\frac{3}{8}''$ back to back, as in girder, their capacity per pair is,

$$4.80 \cdot 15{,}000\left(1 - .006\frac{78}{2.90}\right) = 60{,}400\ \#$$

and the inside pair of angles which will be employed at each reaction should have a thickness not less than $\frac{3}{8}''$.

Spacing of intermediate stiffeners is,

$$200\left(t - \frac{V}{S_c d}\right) = 75 - .000171\ V \text{ with } V \text{ in pounds}$$

$52''$ for 0 to 1; 61.9, 1 to 2; 69.6, 2 to 2.

Composition of Flange

$$A = M/Sh.$$

Net area required at 1 is

$$\frac{19{,}085{,}000}{76 \cdot 15{,}000} = 16.7 \text{ sq.in.};$$

in center panel is

$$\frac{28{,}428{,}000}{76 \cdot 15{,}000} = 25.0 \text{ sq.in.}$$

We may now plot Fig. 93*h*.

Rivet Spacing

	Horizontal Rivets. $\frac{7}{8}''$ Shop Rivets in Bearing on $\frac{3}{8}''$, value 5060 #.	Vertical Rivets. Pairs of $\frac{7}{8}''$ Shop Rivets in Single Shear, Value of pair, 8000 #.
Panel 0–1 spacing.....	$\frac{5060 \cdot 76}{134{,}300} = 2.87''$	$\frac{8000 \cdot 76}{134{,}300} \cdot \frac{20.48}{10.72} = 8.68''$
Panel 1–2 spacing.....	$\frac{5060 \cdot 76}{76{,}700} = 5.02''$	$\frac{8000 \cdot 76}{76{,}700} \cdot \frac{25.07}{15.31} = 13.0''$
Panel 2–2 spacing.....	$\frac{5060 \cdot 76}{31{,}500} = 12.2''$	$\frac{8000 \cdot 76}{31{,}500} \cdot \frac{25.07}{15.31} = 31.7''$

Web Splices

Maximum length of web plates obtainable is about 33 feet. We will locate splices at center. It is customary to put not less than two rows of rivets on each side and to keep them in horizontal rows for multiple punching. This requirement will be found to give ample strength for shear only. Splice plates are commonly made the same thickness as the flange angles and here too there is excess capacity.

Angle Splices

A girder of this length need have no splices in flange. To illustrate method, however, let us splice angles, one on either

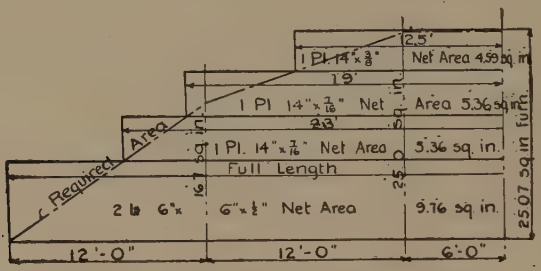

Fig. 93*h*.—Flange Diagram for Plate Girder.

side of the center as shown in Fig. 93*f*. We will use two angles, the one nearest the splice being figured to carry the entire load. The flange angles are:

2 Ls $6'' \times 6'' \times \frac{1}{2}''$. Gross area 11.50 sq.in., net 9.76. For splices we will employ 2 Ls $6'' \times 6'' \times \frac{9}{16}''$ with $\frac{1}{2}''$ cut off each end to fit interior of $6'' \times 6'' \times \frac{1}{2}''$ flange angles.

2 Ls $5\frac{1}{2} \times 5\frac{1}{2} \times \frac{9}{16}''$. Gross area 11.74 sq.in., net 9.77.

Number of shop rivet shears necessary is,

$$4.88 \cdot 15{,}000/0.442 \cdot 9000 = 19,$$

and this number should be provided in addition to those required to carry the horizontal force.

Bottom Laterals. (Fig. 93*f*.)

Transverse loads per lin.ft.

Wind, $40 \cdot 7 = 280$# for both girders.

Stiffening top chord is $22^* \cdot 15{,}000/30 \cdot 60 = 183$# for one girder.

Stress in end panel is $650 \cdot 12 \cdot 2 \cdot 1.25 = 19{,}500$#.

Area required is $19{,}500/15{,}000 = 1.30$ sq.in.

Use the minimum, one L, $3'' \times 3'' \times \frac{5}{16}''$, gross area 1.78, net 1.51.

The deduction due to own weight is less than that due to rivet holes and the two do not occur simultaneously.

In the author's opinion, an end floor beam would be much better on account of the additional stiffness which it gives the end of the girder.

Bearings

Roadway stringer, $11{,}800/400 = 30$ sq.in.; use $6'' \times 10''$.
Sidewalk stringers, $3{,}240/400 = 8$ sq.in.; use $6'' \times 8''$
Girders $148{,}000/400 = 370$ sq.in.; use $20'' \times 20''$.

Actual Weights

A rough computation of some of the weights has been made and compared with estimated values.

	Computed	Estimated
1 main floor beam	4700#	5700#
1 floor beam bracket	720#	800#
1 girder	20,500#	18,000#
All laterals	2500#	

The discrepancy between formula and actual weight in the case of the girder is largely due to the use of the approximate formula.

* Average area top chord.

EXAMPLES. Dimensions in feet.

THROUGH OR HALF THROUGH PLATE GIRDER BRIDGES WITH PLANK FLOOR

E. N., Engineering News. E. R., Engineering Record.

Period-ical.	Vol.	Page.	Span.	Spacing of Girders.	Depth b. to b. Angles.	Ends.	Location.
E. N.	26	257	60.0	33.3	Variable	Skew	Brookline, Mass.
DECK SOLID FLOOR PLATE GIRDER BRIDGES							
E. R.	40	451	52.0	27.4	4.1	Square	Philadelphia, Pa.
E. R.	32	471	103	22.0	9.5	Skew	Troy, N. Y.
THROUGH OR HALF THROUGH SOLID FLOOR PLATE GIRDER BRIDGES							
E. R.	62	465	64.6	32.5	7.0	.....	Buffalo, N. Y.
E. N.	28	63	72.0	26.3	7.0	.....	Cumberland, Md.
E. R.	57	260	72.6	56.0	7.5	Skew	Philadelphia, Pa.
E. N.	27	205	88.0	40.0	8.0	Square	Rockford, Ill.
E. R.	61	662	106.5	36.0	9.5	Skew	Jersey City, N. J.
E. N.	34	174	119	....	9.5	Skew	Philadelphia, Pa.

Prob. 93. Write a bill of material for bridge designed above.

Art. 94. Determination of Deflection of Plate Girders.

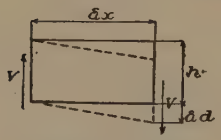

FIG. 94a.—Deflection Due to Shear.

This problem involves methods more complicated than the ordinary beams considered in Applied Mechanics.

The deflection is in two parts; that due to shear and that due to moment. The first is comparatively unimportant and is often neglected. Its value may be computed by the equation:

$$\delta d = V\,\delta x/thF,$$
$$d = \int V\,\delta x/thF,$$

where F is the shearing modulus of elasticity; V, shear; t and h, breadth and depth of web. If there be a uniform load, $V = wL/2 - wx$, where w is the load per lin. unit, L, the span, and x the distance from the left abutment. Substituting,

$$d = \frac{1}{thF}\int_0^{L/2}\left(\frac{wL}{2} - wx\right)\delta x = \frac{wL^2}{8thF},$$

which holds for uniform thickness of web only. For variable thicknesses, treat like deflection due to moment. A mean value of F for steel is 12,000,000 lbs. per sq.in.

As an example, let us find the deflection due to shear of the plate girder in preceding article when loaded with 5400 lbs. per lin.ft., approximate equivalent uniform load.

$$d = \frac{450 \cdot 720 \cdot 720}{8 \cdot 0.375 \cdot 78 \cdot 12{,}000{,}000} = .083'', \text{ about } 5/64''.$$

There are five ways of obtaining the deflection due to moment; the first two are rough approximate methods, the latter three are exact:

(1) *Considering as a Uniformly Stressed Beam.*
(2) *Estimating Average Moment of Inertia.*
(3) *Use of Calculus.*
(4) *Principle of Work.*
(5) *Mohr's Graphical Method.*

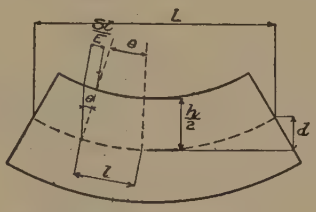

FIG. 94*b*.—Deflection of Beam.

In obtaining moment of inertia, use gross area rather than net, because it more nearly represents the average strength. Also while the web may be weakened by splices for shear only, it is probably better to include it in moment of inertia, since it must carry considerable moment even though girder has been computed by approximate method.

(1) *Considering as a Uniformly Stressed Beam*

Let S be the stress at outside fiber and R the radius of curvature of bent beam.

From similar triangles,

$$\frac{Sl}{E} : \frac{h}{2} :: l : R \quad \text{or} \quad R = \frac{hE}{2S}.$$

From Railroad Engineering:

$$d = \frac{L^2}{8R} = \frac{SL^2}{4hE}.$$

S should not be taken at its full allowable value, but an estimate made of the average unit stress on gross area with the web carrying its part. Let us compute for same problem, a rough value of the deflection, allowing 10,000 lbs. per sq.in. as an average. This is obtained by considering net as 83% of gross and that average stress on net area is 12,000 lbs. per sq.in.

$$d=\frac{10{,}000\cdot 720\cdot 720}{4\cdot 78\cdot 30{,}000{,}000}=0.55''.$$

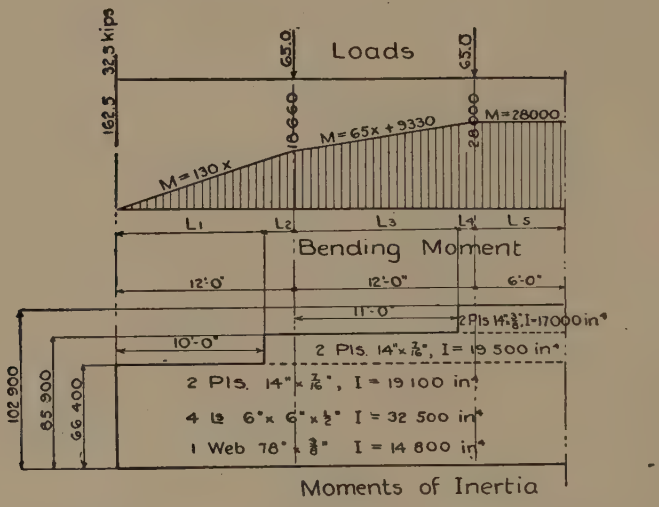

FIG. 94*c*.—Bending Moments and Moments of Inertia for Plate Girder.

(2) *Estimating Average Moment of Inertia*

Here we use the ordinary deflection formula, estimating an average value for I. Note that more weight must be attached to the I at center on account of its greater effect upon the deflection. In above problem, let us use I for web, four angles, and four, $14''\times 7/16''$ cover plates, all gross, equals 86,000.

$$d=\frac{5}{384}\frac{Wl^3}{EI}=\frac{5}{384}\frac{5400\cdot 60\cdot 720\cdot 720\cdot 720}{30{,}000{,}000\cdot 86{,}000}=0.61''.$$

(3) *Use of Calculus*

We may divide beam up into different lengths, for each of which I is a constant and a single equation holds for the moment. The constants of integration may be obtained by the use of the usual conditions, and, in addition, the fact that the tangent of angle of inclination and deflection as given on either side at each junction of the lengths must be identical. Let us apply this method to the same problem. We will keep everything in kips and inches.

For our problem we have considered bottom cover plates to extend to abutment, and have made some alterations in lengths.

$$EI\frac{\delta^2 y}{\delta x^2}=M.$$

For L_1, this becomes $15{,}320{,}000\frac{\delta^2 y}{\delta x^2}=x.$ . . . . . (1)

L_2, this becomes $19{,}820{,}000\frac{\delta^2 y}{\delta x^2}=x,$ . . . . . (2)

L_3, this becomes $39{,}640{,}000\frac{\delta^2 y}{\delta x^2}=x+143.5,$ . . (3)

L_4, this becomes $47{,}500{,}000\frac{\delta^2 y}{\delta x^2}=x+143.5,$ . . (4)

L_5, this becomes $110{,}250\frac{\delta^2 y}{\delta x^2}=1.$ . . . . . (5)

Integrating once,

For L_1, this becomes $30{,}640{,}000\frac{\delta y}{\delta x}=x^2+C_1.$ . . (11)

L_2, this becomes $39{,}640{,}000\frac{\delta y}{\delta x}=x^2+C_2.$ . . (12)

L_3, this becomes $79{,}280{,}000\frac{\delta y}{\delta x}=x^2+287x+C_3.$ (13)

L_4, this becomes $95{,}000{,}000\frac{\delta y}{\delta x}=x^2+287x+C_4.$ (14)

L_5, this becomes $110{,}250\frac{\delta y}{\delta x}=x+C_5.$ . . (15)

From symmetry, $\frac{\delta y}{\delta x}=0$, when $x=360$, hence $C_5=-360$.

When $x=$

288, $\frac{\delta y}{\delta x}$ in (14) and (15) must be equal, hence $C_4=-227{,}600$.

276, $\frac{\delta y}{\delta x}$ in (13) and (14) must be equal, hence $C_3=-215{,}500$.

144, $\frac{\delta y}{\delta x}$ in (12) and (13) must be equal, hence $C_2=-\ 97{,}700$.

120, $\frac{\delta y}{\delta x}$ in (11) and (12) must be equal, hence $C_1=-\ 78{,}800$.

Substituting these values and integrating once more,

L_1, $y=.000000010 9x^3-.00257x+C_6$. . . . . (21)
L_2, $y=.000000008 4x^3-.00246x+C_7$. . . . . (22)
L_3, $y=.000000004 2x^3+.000001 81x^2-.00271x+C_8$. (23)
L_4, $y=.000000003 51x^3+.000001 51x^2-.00239x+C_9$. (24)
L_5, $y=.00000454x^2-.00326x+C_{10}$. . . . . (25)
$y=0$, when $x=0$, hence $C_6=0$.

When

$x=120$, y in (21) and (22) must be equal, hence $C_7=-.0088$.
$x=144$, y in (22) and (23) must be equal, hence $C_8=+.0022$.
$x=276$, y in (23) and (24) must be equal, hence $C_9=-.0485$.
$x=288$, y in (24) and (25) must be equal, hence $C_{10}=+.035$.
$x=360$, y is a maximum, equals $0.550''$.

Equations now become,

L_1, $y=.000000010 9x^3-.00257x$ . . . . . (31)
L_2, $y=.000000008 4x^3-.00246x-.0088$. . . . . (32)
L_3, $y=.000000004 2x^3+.000001 81x^2-.00271x+.0022$. . . (33)
L_4, $y=.000000003 51x^3+.000001 51x^2-.00239x-.0485$. . (34)
L_5, $y=.00000454x^2-.00326x+.035$. . . . . (35)

(4) *Principle of Work*

By this method the deflection at any point may be obtained.

$$d = \int_0 Mm\delta x/EI,^*$$

where M represents moment of loads actually on the beam,
m represents moment of load unity acting at a point and in the direction in which the deflection is desired.

As an example of its use let us compute deflection at center in the same problem.

$$\int Mm\delta x/EI \text{ for both ends,}$$

for

$$L_1 \text{ is } 2\int_0^{120} \frac{130x \cdot 0.5x\,\delta x}{30{,}000 \cdot 66{,}400} = 0.037''.$$

$$L_2 \text{ is } 2\int_{120}^{144} \frac{130x \cdot 0.5x\,\delta x}{30{,}000 \cdot 85{,}900} = 0.021''.$$

$$L_3 \text{ is } 2\int_{144}^{276} \frac{(65x+9330)0.5x\,\delta x}{30{,}000 \cdot 85{,}900} = 0.253''.$$

$$L_4 \text{ is } 2\int_{276}^{288} \frac{(65x+9330)0.5x\,\delta x}{30{,}000 \cdot 102{,}900} = 0.030''.$$

$$L_5 \text{ is } 2\int_{288}^{360} \frac{28{,}000 \cdot 0.5x\,\delta x}{30{,}000 \cdot 102{,}900} = 0.212''.$$

Total is $.037+.021+.253+.030+.212=0.553''$, in substantial agreement with the results obtained by the use of the calculus. It is undoubtedly simpler and quicker if deflection is to be found at one point only.

(5) *Mohr's Graphical Method*

In Fig. 94*b*,

$$\theta = 2Sl/hE.$$

But

$$S = Mh/2I.$$

Substituting,

$$\theta = Ml/EI.$$

* Merriman's "Mechanics of Materials," Art. 124.

Or in words, the angle between two adjacent sections is equal to average moment multiplied by the length between said sections and divided by the product of the modulus of elasticity and moment of inertia of section.

Now if we select two sections and lay off to any scale, one inch equals so many inch^2 pounds, Ml vertically and EI horizontally, we would have the angle between tangents to curve of deflection at beginning and end of section. And if, as in Fig. 94*d*, we lay off on the same vertical line, Ml for next section, the angle between first and final lines would be angle between deflected positions. If now the EI has changed, the only difference it makes in our work is shown in Fig. 94*e*. We proceed then as follows, Fig. 94*f*:

FIG. 94*d*.—Mohr's Diagram for Uniform Section.

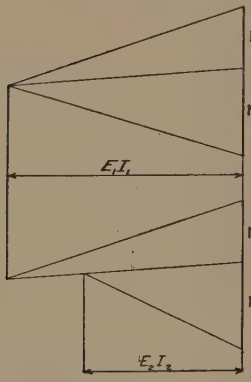

FIG. 94*e*.—Mohr's Diagram for Variable Section.

(*a*) Draw moment diagram of loading for which deflection is desired.

(*b*) Divide into not less than twenty parts, bringing points of division at change of I.

(*c*) Compute Ml for each of these divisions and consider as applied at the center of gravity of the area.

(*d*) Lay off in order on a vertical line, M_1l_1, M_2l_2, etc.; if signs change, lay off in opposite direction.

(*e*) If we plotted EI to same scale as Ml, we should have actual angle, but it is usually better to distort this. If scale of EI be made the same fraction of the scale of Ml that length of moment diagram is of actual length of beam, the resulting deflections will be full size.

(*f*) Lay off EI horizontally on the side which will make beam bend in right direction; if the tangent to the curve of deflection is known at any point, corresponding ray should be drawn parallel thereto. For example, in our problem, girder is horizontal at center. If no such point be known, we will assume direction of any ray. Care must then be taken to measure the final deflection vertically.

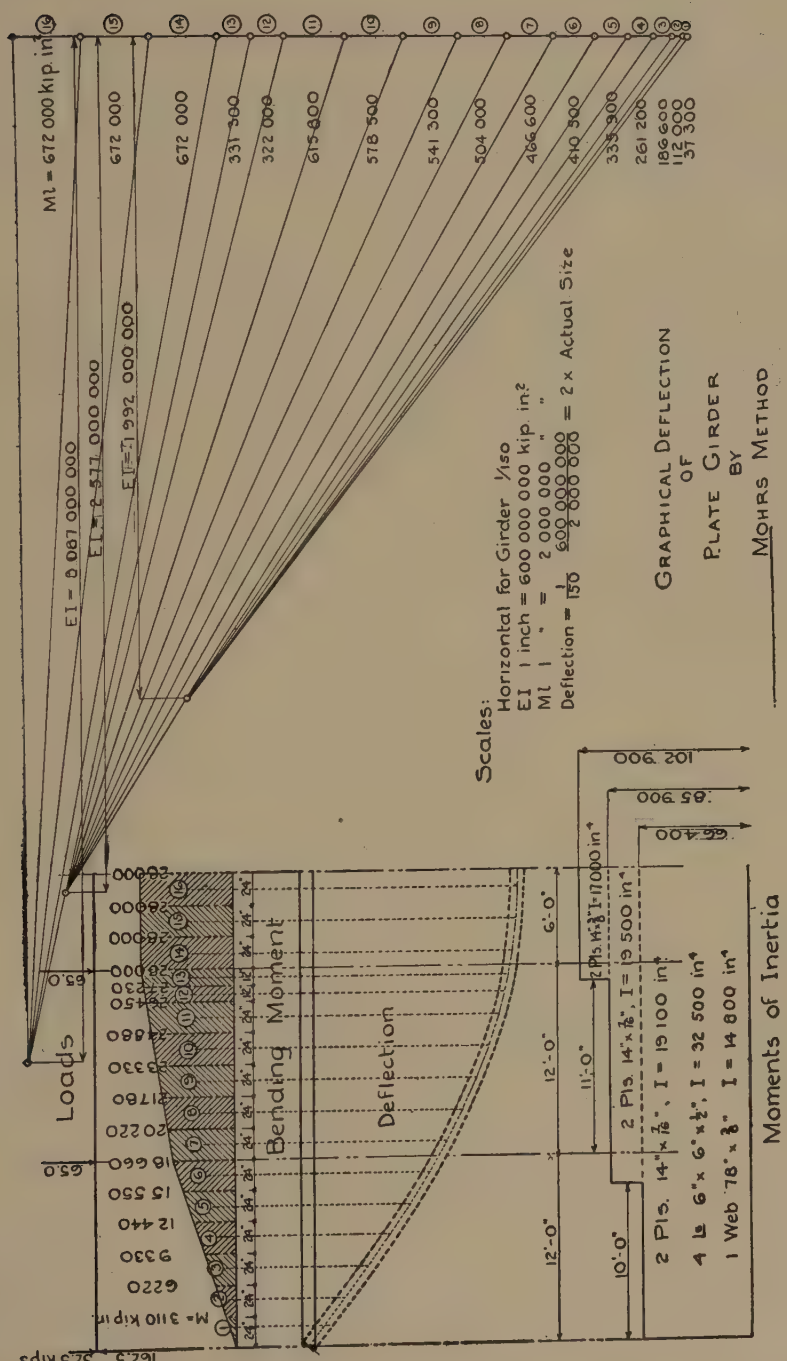
Ml = 672 000 kip. in.
672 000
672 000
331 300
322 000
615 800
578 500
541 300
504 000
466 600
410 500
335 900
261 200
186 600
112 000
37 300
EI = 3 087 000 000
EI = 2 577 000 000
EI = 1 992 000 000
Scales:
Horizontal for Girder 1/150
EI 1 inch = 600 000 000 kip. in.²
Ml 1 " = 2 000 000 " "
Deflection = 1/150 600 000 000 / 2 000 000 = 2 x Actual Size
GRAPHICAL DEFLECTION OF PLATE GIRDER BY MOHRS METHOD
Loads
Bending Moment
Deflection
Moments of Inertia
M = 3110 kip in.
6220
9330
12 440
15 550
18 660
20 220
21 780
23 330
24 880
26 450
27 230
28 000
28 000
28 000
28 000
28 000
65.0
65.0
162.5
32.5 kips
6'-0"
12'-0"
12'-0"
11'-0"
10'-0"
2 Pls. 14"x3/8", I = 17 000 in⁴
2 Pls. 14"x7/16", I = 19 500 in⁴
2 Pls. 14"x7/16", I = 19 100 in⁴
4 Ls 6"x6"x1/2", I = 32 500 in⁴
1 Web 78"x3/8" I = 14 800 in⁴
102 900
85 900
66 400

FIG. 94*f*.

(*g*) Proceed as though *Ml* were loads to draw the equilibrium polygon. The vertical ordinates then represent to scale not moments but deflections.

Students should be able to thoroughly understand this after following through Fig. 94*f* which solves the same problem graphically.

REFERENCE

American Society of Civil Engineers, Vol. 51, p. 1.

Prob. 94. Obtain deflection of plate girder of Arts. 89 and 90 when load on each girder is 4200 lbs. per lin.ft., uniformly distributed. Use method (*3*), (*4*), or (*5*), checking by (*1*) or (*2*).

CHAPTER VII

STEEL VIADUCTS AND ELEVATED RAILROADS

Art. 95. General

THE term "viaduct" is usually applied to an elevated road. For steel structures, this means a bridge very much like those which rest on abutments and piers, but supported by steel columns. We will now consider the design of the latter with the bracing necessary to hold them to their duty and to carry wind, tractive, and centrifugal loads. They may be classified as:

(1) High Viaducts.
(2) Low Viaducts.
(3) Viaducts with Partial Bracing.

The approximate weight of the towers of a viaduct in lbs. when economically arranged is:

$$12Wh/S,$$

where W is the total weight on towers in lbs.;
h, height, base of rail to top of masonry, in feet;
S, allowable unit stress in short strut in lbs. per sq.in.

If w represents uniform load per foot for entire bridge, the weight of towers per lin.ft. becomes:

$$12wh/S.$$

The weight of the girders and bracing per lin.ft. is:

$$18l\sqrt{w/S}, \qquad \text{Art. 88,}$$

where l equals the span in feet. Let us take for a railroad the average values, $w=8000$, $S=10{,}000$, $l=50$, and the total then becomes:

$$9.6h+800 \text{ for single track,}$$

$$19.2h+1600 \text{ for double track.}$$

FIG. 95.

The amount of fill is, in cubic feet, assuming a width at base of rail of 12 feet for single track, and 26 for double, with slopes of $1\frac{1}{2}$ horizontal to 1 vertical,

$$12h+1.5h^2 \text{ for single track,}$$

$$26h+1.5h^2 \text{ for double track.}$$

Allowing $3\frac{1}{2}$ cents per pound for steel and 1 cent per cubic foot for fill, these become in cents per linear foot:

Cost of Viaduct.	Cost of Fill.	Tracks.
$34h+2800$	$12h+1.5h^2$	Single
$68h+5600$	$26h+1.5h^2$	Double

Whence we derive that when h equals 51 for single track and 76 for double track, costs are equal. Above these limits, steel should be cheaper. But we have not considered sinking fund, maintenance charges, and cost of abutments and pedestals, all of which would make steel more expensive. Also costs, loads, and many other factors vary with each individual case. Fill should be employed up to 40 feet. From 40 to 100 feet in height, make careful preliminary estimates of cost based on designs for each method of crossing.* It will seldom be expedient to fill more than 100 feet.

The locating engineer should strive to avoid high viaducts where masonry footings will be expensive. The maximum span that can be erected without falsework is about 100 feet, and average will be much less, necessitating many pedestals. Of course we might have long spans with falsework, but the latter is particularly costly for high structures. Still another solution of this problem is furnished by the use of cantilever bridges (Vol. III), arches (Vol. III), or by simple trusses, all being erected by the cantilever method. (Art. 49.)

High viaducts carrying a roadway are rather unusual. They are limited largely to short deep valleys in or near cities. In other locations, the grade and alignment may generally be so chosen as to avoid them.

* In the cut-off line of the Delaware, Lackawanna, and Western, there is a fill of 75 to 110 feet for a distance of three miles. See Railway and Engineering Review, Vol. 51, p. 67.

We have then:

(*1*) *High Viaducts*, usually railroad bridges, with spans less than 100 feet and heights over 40 feet. Generally located where land is cheap and good foundations can be had near the surface.

(*2*) *Low Viaducts*, where property is valuable or required passage ways frequent or both. Spans will vary with requirements but will seldom exceed 100 feet. This condition occurs often in track elevation and in the approaches to city bridges.

(*3*) *Viaducts* with *Special Bracing* should be employed where most of the spans are to be used as passage ways. The best example of such a condition is an *elevated railroad* in a city street.

We shall now consider as illustrative of these types the following:

High Viaducts,	For Railroads,	For Highways;
Low Viaducts,	Track elevation,	Bridge approaches;
Viaducts with special bracing,	Elevated Railroads.	

The principles developed in the consideration of the above will suffice for all viaducts. Methods of erection have been treated in Art. 49. Loads will be those already given. Transverse and longitudinal forces—wind, tractive, and centrifugal—become very important. About the only change in the specifications, Arts. 88 and 92, will be to insert the following clause:

Structure shall preferably be so designed as to eliminate uplift. Should it be possible, full provision must be made for its effects at all points.

Prob. 95. Assuming above prices for fill and steel, also that maintenance charges are zero for former; that maintenance, depreciation, and so forth, for the latter will amount to replacement every forty years, compute relative annual costs for a single track fill 80 feet high with money at 4%.

Art. 96. High Viaducts for Railroads

The typical steel viaduct for railways consists of deck plate girders, as described in last chapter, but resting upon posts rather than shoes. "Lattice girders," small riveted trusses, occasionally replace the intermediate plate girders. As already mentioned, the maximum span for the usual method of erection is about 100 feet. For conditions demanding more, we use truss bridges, resting upon similar posts. In either case, these posts must be braced together transversely to the rails. A pair so braced is called a "bent," Figs. 96*h* to 96*k*.

FIG. 96*a*.—Salisbury Viaduct. Western Maryland R. R.

Provision for tractive forces require longitudinal bracing between bents, which is usually placed in every other panel. These panels are somewhat shorter than the others to economize in bracing. Amount varies from 30 to 60 feet, increasing somewhat with the height in order to prevent uplift. Two bents so braced together are termed "towers." Intermediate spans are about twice as long, 60 to 100 feet. Theoretically, both lengths should increase with the depth. They are, however, usually kept constant for a given structure, the economy in shop work and engineering being greater than saving in material. A typical viaduct will appear about as shown in Fig. 96*b*. Here pedestal footings should be brought to such

a height that bents (3), (4), (5), and (6) can be made alike. Furthermore, the upper part of all bents should be identical.

The girders on top of the towers should be fixed at both ends. Intermediate spans usually have one end fixed and

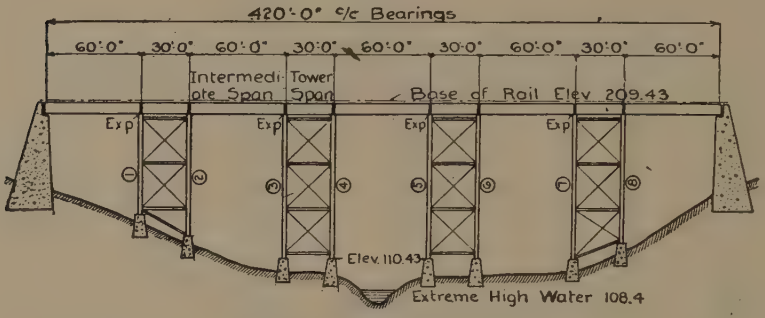

FIG. 96*b*.—A Typical Viaduct.

the other free. However, it will do no harm if expansion joints are placed only in every other intermediate span. This expansion joint is made by bolting the cap plate of the column to the sole plate of the girder, holes in the latter being slotted the proper amount. For fixed ends, the same connection is made except that rivets are driven into round holes to fasten

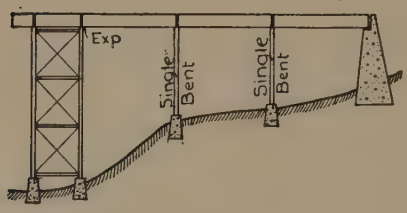

FIG. 96*c*.
Viaduct with Single Bent.

FIG. 96*d*.
Pin Joints in Rocker Bent.

the two, Fig. 96*b*. Where viaducts are on a grade, bents usually remain vertical, and the cap plates are beveled to correspond with the pitch of the girder and track.

For small elevations, single bents without longitudinal bracing may be employed, Fig. 96*c*. There is a good deal of

spring in a column like this, but it is better to avoid the secondary stresses, Art. 109, by pin joints as shown in Fig. 96*d*, unless the post is a long one and needs the fixed ends to strengthen it as a column.

As already noted, tower span is usually less than intermediate. We might employ two different depths, each being the most

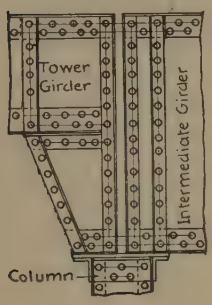

FIG. 96*e*.

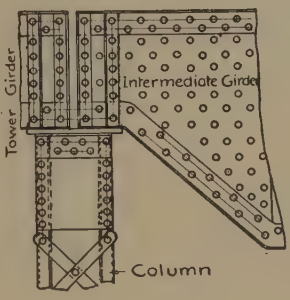

FIG. 96*f*.

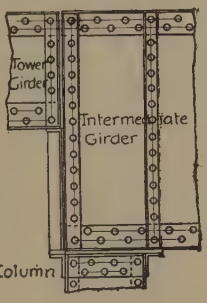

FIG. 96*g*.

Methods of Handling Different Depths of Girders at the Columns.

economical for its own span. The connection at posts could then be handled as shown in Figs. 96*e*, 96*f*, or 96*g*. However, each of these involve considerable additional shop work and it is better to use a mean economical depth throughout and this is the usual design.

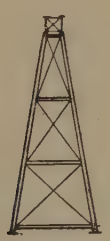

FIG. 96*h*.

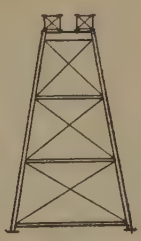

FIG. 96*i*.

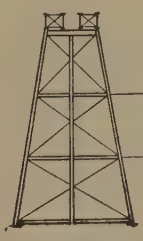

FIG. 96*j*.

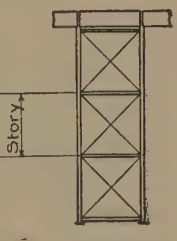

FIG. 96*k*.

Different Types of Transverse Bents. Typical Tower.

Fig. 96*h* shows typical form for transverse bent for single track; 96*i* and 96*j*, the same for double track, the former being better; 96*k*, the arrangement of longitudinal bracing for a tower. The points of subdivision may or may not be coin-

cident; the former makes a stronger column, but involves some difficulty in the two simultaneous connections; it is, however, the best method. The part between two points of subdivision is called a story; its average height is 30 feet. This makes angle of bracing about 45°, giving best efficiency, and it supports the columns at fairly frequent intervals. It is often somewhat influenced by the desire to make upper parts of all bents alike; in Fig. 96*b*, two upper stories of all bents should be duplicates.

Transversely, width at the top should be that of the outside girders; it may, however, be somewhat more. The bevel

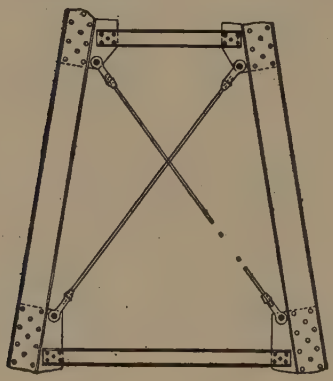

FIG. 96*l*.
Bracing with Adjustable Members.

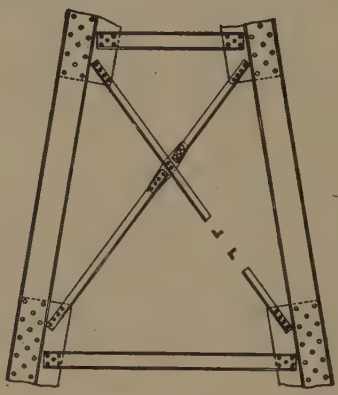

FIG. 96*m*.
Bracing with Stiff Tension Members.

of the inclined legs is such as to prevent uplift at masonry; this varies from $1\frac{1}{2}$ to 3″ per vertical foot. 2″ is quite common; it is easy to compute and lay out and will usually be enough to prevent uplift on tangent; it has no other advantages. There should be no bevel longitudinally, that is, all bents should be in a vertical plane. If another slope be introduced, it gives difficult detailing and objectionable bent work.* As already mentioned, the longitudinal distance between bents varies from 30 to 60 feet, with 40 feet for an average.

* The angle between the two planes of the longitudinal and transverse bracing then becomes obtuse and must be computed by the aid of the principles of solid trigonometry or laid out graphically. See Vol. III.

Bracing

There are three methods of bracing. They are shown in the figures as applied to viaducts.

(*1*) *Bracing with Adjustable Members*, Fig. 96*l*. These are usually rounds or squares, upset at ends and threaded for clevis nuts. While theoretically there is an interference at center, each may be bent a little to let the other pass. At this intersection, they should be fastened together by wire to prevent rattling. They do not make a stiff structure, are likely to work out of adjustment, and are used only on inferior work.

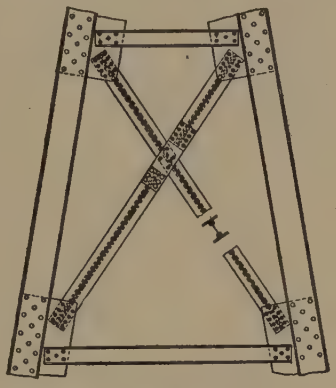

Fig. 96*n*.
Bracing with Compression Members.

(*2*) *Bracing with Stiff Tension Members*, Fig. 96*m*. Here, as in (*1*), members are designed like the counters of a truss to carry tension only. Nevertheless they are built of shapes which can stand a little compression and are stiff enough so that they will not rattle. To prevent their buckling out of line, fabricated length should be made .0005 of the computed length short (1/16″ for every 10 feet), thus putting an initial stress into the laterals.

This type of bracing is made by single angles, which cannot be bent past at their intersection like rods. However, one angle may run through while the other is spliced across, Fig. 96*m*, all may be spliced across, Fig. 96*o*, or one angle may be turned in while other is turned out. They should always be riveted here to reduce stresses due to own weight.

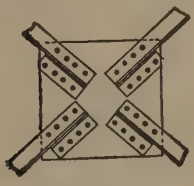

Fig. 96*o*.
Bracing at Intersection.

Sometimes, as in viaducts, bracing is desirable in two planes * and then two angles are employed. They may then be battened or latticed together, thus to some extent making

* E.g., in viaducts, angles are fastened to lateral plates on both sides of channels.

them like type (3). When so designed, consider methods of erection with great care.

This style is economical and widely used. For heavy and rapidly moving loads, it is not as satisfactory as

Type (3). *Bracing with Compression Members*, Fig. 96*n*. Here the diagonals are computed and designed to carry either tension or compression. Four angles, latticed, or two channels, latticed, are common. Methods of handling intersections are similar to those for Type (2). This is the best method, also most expensive.

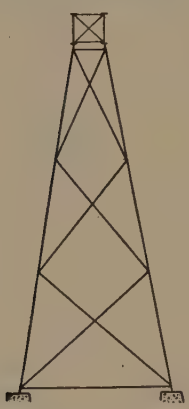

FIG. 96*p*.

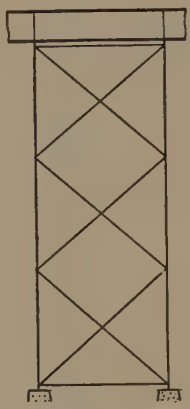

FIG. 96*q*.

Viaducts without Intermediate Struts.

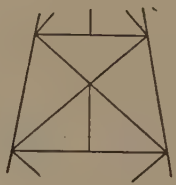

FIG. 96*r*.

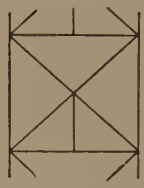

FIG. 96*s*.

Supports for Long Struts.

We shall hereafter find frequent occasion to refer to above classification. (2) and (3) are found in the better class of viaducts. Sometimes where the latter is used, intermediate horizontal struts are dispensed with and forms of Figs. 96*h* and 96*k* become those of Figs. 96*p* and 96*q*.

STRUTS

Struts should be placed as near top and bottom as practicable. Center lines for all struts should intersect those of columns and diagonals at the same points. This rule is sometimes partially disregarded to secure compact connections. To lessen stresses due to own weight, vertical members are often inserted as seen in Figs. 96*r* and 96*s*. Struts are commonly of four latticed angles or two latticed channels.

Posts

Many different sections are employed here; two channels latticed, two channels and one cover plate, three Is, two channels and one I, and Z bar. The first is most frequent and is typical of viaduct work. The webs of the channels are placed in planes transverse to the rails.

Column may be in one-or two-story lengths and should be spliced

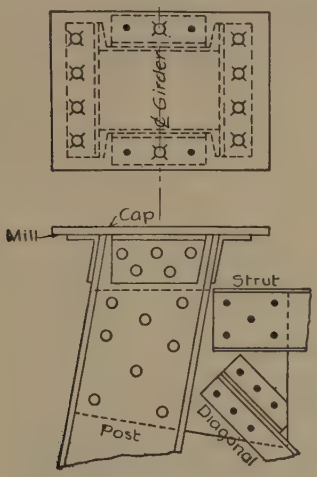

FIG. 96*t*.—Column Cap.

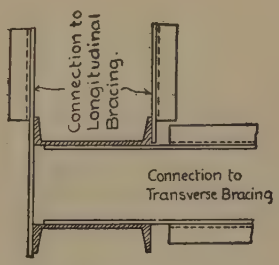

FIG. 96*u*.—Intermediate Connection.

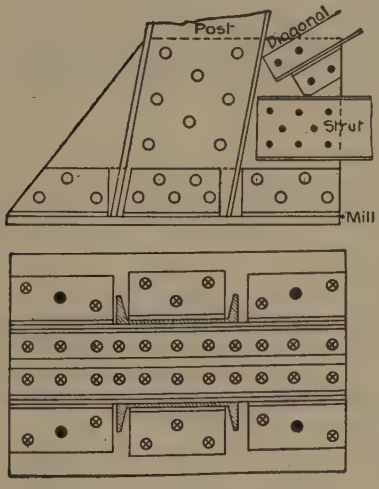

FIG. 96*v*.
Column Base.

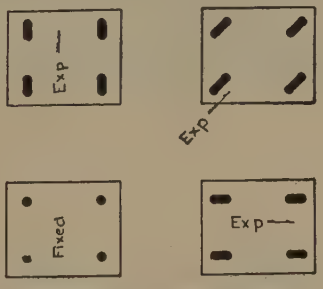

FIG. 96*w*.—Slotting of Holes for Expansion of Viaducts.

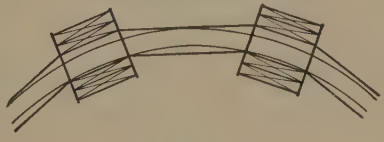

FIG. 96*x*.
Arrangement of Viaduct for Curved Track.

as near joints as practicable. Column ends are milled and two or three rows of rivets placed in splice plates on either side of splice and on four sides of post.

The usual simple design for cap is seen in Fig. 96*t*. The cap plate should be $\frac{3}{4}''$ to $1''$ in thickness. It is fastened to cap angles by stitch rivets, countersunk in cap plate where necessary to clear sole plate of girder. Surfaces along which expansion takes place must be planed. Cap angles are about $6''\times4''\times\frac{1}{2}''$ with longer leg vertical.

Fig 96*u* shows a typical mid-column connection. It should be noted that plates are shop riveted to post.

Fig. 96*v* shows a typical base. Plate should be $\frac{3}{4}''$ to $1''$

FIG. 96*y*.—Genessee Viaduct, Erie R. R.; McClintic-Marshall Construction Co., Rankin, Pa.

thick and rest on a masonry plate of equal size but with holes for bolts only. If there is any portion of the base not well stiffened by gussets, it must not be counted as available area.

Sometimes, more particularly for very high viaducts, provision is made for expansion by means of slotted holes in base. Fig. 96*w* shows diagrammatically how this is done. The desirability of such an arrangement is doubtful.

Where track is curved, locate girders as already explained in Art. 83. Make plane of each bent in a tower, parallel to a radial line at the center of tower, Fig. 96*x*.

Examples

(Listed in order of height)

Dimensions are in feet,

Abbreviations,

E. N., Engineering News.
E. R., Engineering Record.
C. E., Canadian Engineer.
E. C., Engineering Contracting.
A. S., American Society of Civil Engineers.
W. S., Journal of the Western Society of Civil Engineers.

Periodical.	Vol.	Page.	Base of Rail to Masonry.	Tower Spans.	Intermediate Spans.	Location.
E. R.	43	26	320	40	60–120	Gotkeik, Burmah.
E. N.	45	147				
E. N.	21	449	314	32	32–80	Loa, Bolivia.
E. N.	62	324	294	67	99–167	Lethbridge, Can.
E. N.	24	7	285	38	61	Kinzua, Pa.
A. S.	46	21				
E. N.	29	2	270	35	35–185	Pecos, Texas.
E. N.	64	471				
E. N.	66	288				
W. S.	16	334				
E. R.	55	104	229	30–40	30–75	Las Vacas, Guatemala.
E. R.	51	120	...	.....	.....	Portage.
E. R.	56	7	177	40	80	Modna Creek, N. Y.
E. R.	49	120	...	38	75–219	Des Moines, Iowa.
		158				
E. N.	45	415				
		466				
E. N.	69	830	156	32	32–80	Boone, Iowa.
E. N.	21	471	...	30	30–60	Buffalo Trace, Ky.
E. R.	57	205	149	40	60–160	Cape Rouge, Can.
E. R.	53	441	...	15–30	30–270	Rondout, N. Y.
E. R.	54	47				
C. E.	25	350	...	30–40	56–150	Boucanne, Quebec.
E. C.	39	691				
E. R.	56	238	...	40	80–120	Genesee River, N. Y.
E. R.	63	166	...	30–40	60–92	Salisbury, Pa.
E. R.	53	612	93	30	51–80	Ashtabula, Ohio.
E. R.	46	266	91	30	60	18 Mile Creek.
E. N.	43	234	88	25	48	Grasshopper Creek, Ill.
E. R.	61	330	84	30	60–100	Shepherdstown, W. Va.
E. N.	53	597	83	60	41–75	Horse Run, Ky.
E. R.	59	239	82	27–40	45–67	Greenville, Me.

REFERENCES

"Details of Bridge Construction, Vol. II, Plate Girders," by Skinner.

"Theory and Practice of Modern Framed Structures," by Johnson, Bryan, and Turneaure.

"Engineering Contracting," Vol. 27, p. 270; Vol. 31, p. 270.

Prob. 96. Suppose the long spans in Fig. 96*c* are 60 feet, and the single bents have heights of 30 and 40 feet. The latter are composed of two posts, each of two channels, 15″ @ 33 ℔ with webs parallel to viaduct. Find flexural stresses produced in them by an expansion of 1″ per 100 feet, assuming both ends as fixed.

Art. 97. Other Viaducts with X Bracing

TRACK ELEVATION

A very common height will be about 20 feet from rail to

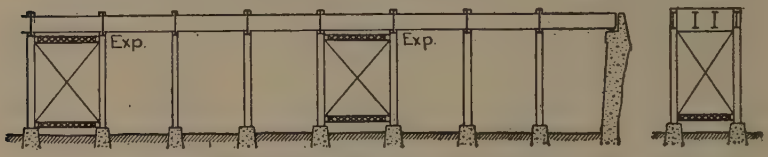

FIG. 97*a*.—Typical Track Elevation—Deck Double Track.

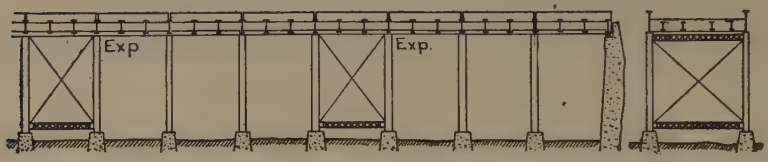

FIG. 97*b*.—Typical Track Elevation—Half Through Double Track.

pedestals, just enough to allow the proper clearance for teams. For such cases, no batter will be required and columns may be made plumb. They will not need bracing at intermediate points, hence a single story will be enough. Economical span is somewhat lessened and towers can be located farther apart. Intermediate posts might be built as rockers, but this is rather unusual. Fig. 97*a* shows typical construction for deck double track; Fig. 97*b* that for half through double track. Unless

width of spans is determined by openings required, spacings of columns will be much less than for high viaducts for economical reasons.

Methods of construction are much like those employed in railroad high viaducts. A single new point appears. The two channel and I section, Art. 56, is very desirable in locations where posts are likely to be hit by a derailed train or a runaway team. Of course the structure should be so designed that ordinary wagon traffic will not hit the post, Fig. 97*d*.

Fig. 97*c*.—Track Elevation—Ninth Street, Philadelphia, Pa.
McClintic-Marshall Construction Co., Rankin, Pa.

The engineer should always consider an alternative design in reinforced concrete.

High Viaducts for Highways

We find here the same general method of construction as for railways. On account of lessened vibration, adjustable members are often employed for diagonals. Due to greater width at top, batter may often be decreased. Small tractive forces render the longitudinal bracing of much less importance. Thirty feet is an average value for tower spans and 50 feet for intermediate.

Deck bridges are best, as there is no question of clearance. Fig. 97*f* shows a good design for the floor system; Fig. 97*g* is also allowable if floor beams and girders are properly braced against overturning; in the illustration, the steel stringers stiffen the former. Fig. 97*h* shows how trusses may be employed to carry these floor beams. For solid floors, stringers of Fig. 97*g* may be omitted and floor beams placed about five feet apart. I-beams are usually strong enough, and, if this is the case, they should be employed on account of low shop cost.

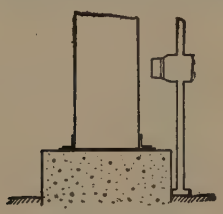

FIG. 97*d*.
Provision against Injury of Post by Vehicles.

LOW VIADUCTS FOR HIGHWAYS

For these, the columns are vertical since necessary width and low tractive forces prevent any danger of uplift. For the same reason, the towers and expansion points may be placed

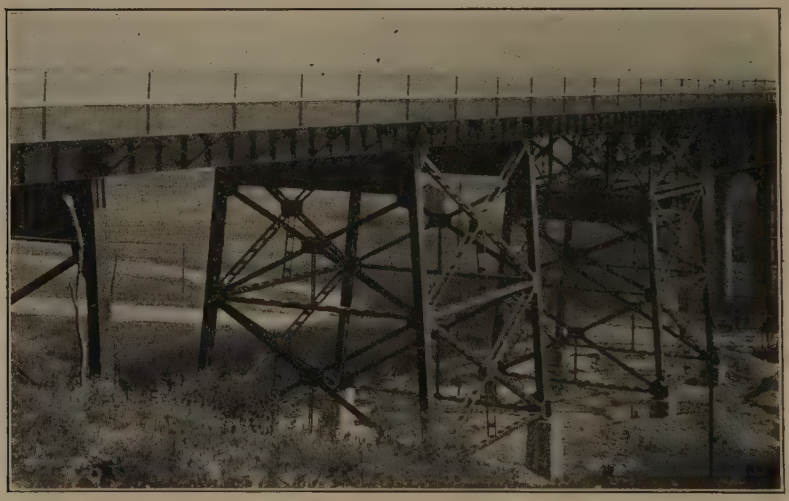

FIG. 97*e*.—Dennison Harvard Viaduct, Cleveland, Ohio.
McClintic-Marshall Construction Co., Rankin, Pa.

farther apart. Short spans are more economical; if conditions underneath will admit, they should be made alike for tower and intermediate spans. Or towers may be omitted and the columns treated as *partially braced*, Art. 99. However, when properly computed, this will not prove economical, and it is better to brace fully if practicable for the given location.

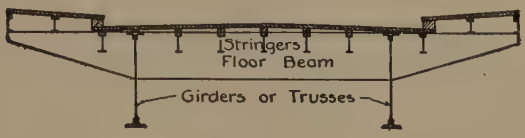

FIG. 97*f*.

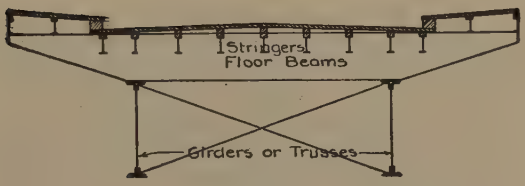

FIG. 97*g*.

Floor Systems for Highway Viaducts.

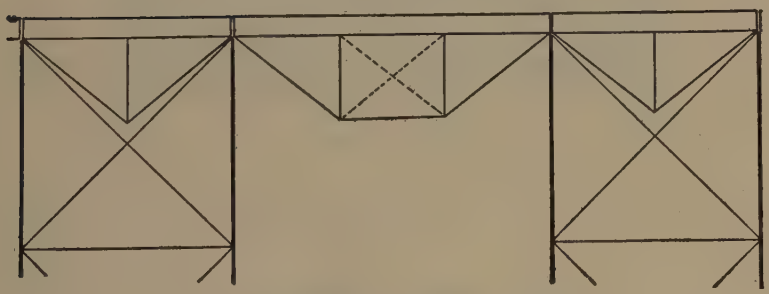

FIG. 97*h*.—Trusses for Highway Viaducts.

There are two general cases:

(*1*) Stringers framing into floor beams, in turn framing into posts.

(*2*) Stringers framing into floor beams, which frame into girders, in turn framing into posts.

(*1*) is preferable for short spans. Occasionally we see these members resting upon supports instead of framing into them. This makes erection easy, but forms a less rigid structure.

Bridge Approaches

These may be either railroad or highway, low or high, or varying in height, and they are often curved and skewed. However, the principles involved are not different from those already developed in preceding text and we will not consider them further.

Fig. 97*i*.—Approach to Queensboro Bridge, on the New York Side.

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Engineering Record, Vol. 52, p. 102; Vol. 54, p. 209; Vol. 60, p. 153; Vol. 61, p. 101.

Highway Viaducts

(Listed in the order of height)

Dimensions are in feet

E. N., Engineering News; E. R., Engineering Record.

Periodical.	Vol.	Page.	Floor to Masonry.	Tower Spans.	Intermediate Spans.	Location.
E. R.	52	239	125	28.5	57	Bellevue, Pa.
E. R.	58	404	104	26.0	26–78	Ben Avon, Pa.
E. N.	41	358	...	65. to	129	Riverside Drive, N. Y.
E. N.	43	334	...	30.0	45–60	Washington, D. C.
E. N.	32	334	...	28.0	42–56	Cleveland, Ohio.
E. N.	38	404	47	30–36	36–135	Edgebrook, Mo. (Elec. Ry).
E. N.	65	8		Varia	ble	Denver, Col.
E. R.	61	736				
E. R.	58	429		20 to	180	Cincinnati, Ohio.
E. R.	25	192 209	25	20 to	130	Duluth, Minn.

Railroad Approaches

"Types and Details of Bridge Construction, Part II, Plate Girders," by Skinner.

Engineering News, Vol. 44, p. 102; Vol. 69, p. 94.

Engineering Record, Vol. 49, p. 544; Vol. 61, p. 572.

Highway Approaches

Engineering News, Vol. 43, p. 301; Vol. 58, p. 323; Vol. 69, p. 94.

Engineering Record, Vol. 38, p. 224; Vol. 39, p. 539; Vol. 41, p. 445; Vol. 56, pp. 567, 623; Vol. 61, p. 572.

Prob. 97. An asphalt-paved bridge in a suburban location 40 feet wide and without sidewalks is to span a valley of triangular cross-section, 600 feet wide and 60 feet in depth at middle, below bridge floor. Assuming fill to stop at 20 feet, decide on proper arrangement and obtain approximate weight.

Art. 98. Viaducts with Special Bracing, Elevated Railroads

The salient differences between elevated railroads and other viaducts are caused by the fact that they usually occupy the streets in populous cities. Where conditions allow, as on a private right of way, they should be braced in both directions and built otherwise like ordinary viaducts, or, better still, located on fill either with or without retaining walls.

The advantages of elevated railroads as compared with street cars are the greater allowable speeds and the elimination of collisions with teams and pedestrians. On the other hand, their disadvantages are:

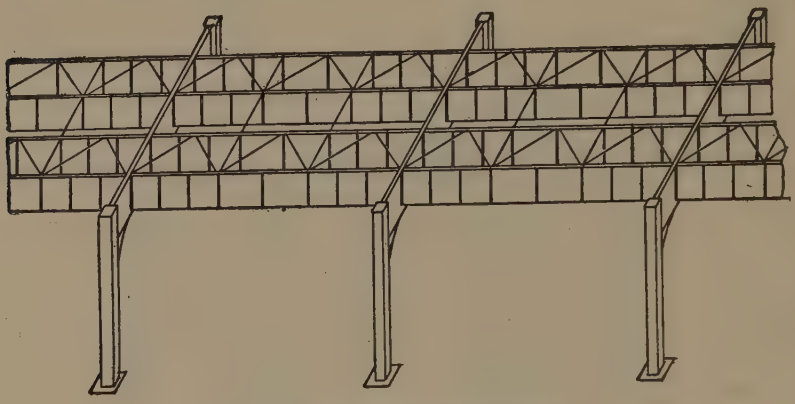


FIG. 98*a*.—Typical Elevated Railroad.

(*1*) Not as easy of access as street cars.

(*2*) They obstruct passage. Least interference is caused when posts are placed on sidewalk near curb.

(*3*) They darken streets and lower stories of adjacent dwellings. "Lattice girders," small riveted trusses, are sometimes substituted on this account for plate girders. They cost about the same but are not considered as satisfactory structurally.

(*4*) Noisy. This condition is somewhat lessened by the use of solid floors.

(*5*) Drop water, oil, and so forth on traffic below. Remedy as in (*4*).

(*6*) They are unsightly. In the Arborway crossing of

the Boston Elevated Roads,* the whole structure was encased in concrete to give a monumental effect.

(7) They are expensive. Rough cost per mile is $500,000 for two tracks and $130,000 for each additional track.

Of recent years, the subway has been the more popular means of rapid transit. It has all the advantages of the elevated and but two of its disadvantages, (1)and (7).

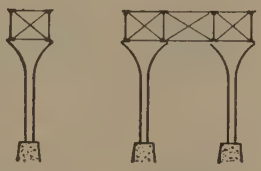


FIG. 98*b*. Unbraced. FIG. 98*c*. Braced.
Isolated Posts.

A typical elevated railroad will then consist of track of rails and ties much like those for surface railroads, these resting on deck girders running parallel to the street and supported by transverse girders, the latter carried by columns. Fig. 98*a* shows this construction with track omitted.

Clearance for the bulk of the width of the street should be not less than 14 feet in height. *Center to center of girders* is usually 5 to 6 feet; *center to center of tracks*, 12 feet.

Usual and most economical span, that is, distance between bents, is 45 to 55 feet. To clear obstructions, much longer spans may be employed, even such as necessitate the use of something besides a plate girder; through trusses or arch bridges for example. Expansion joints should be placed about 200 feet apart.

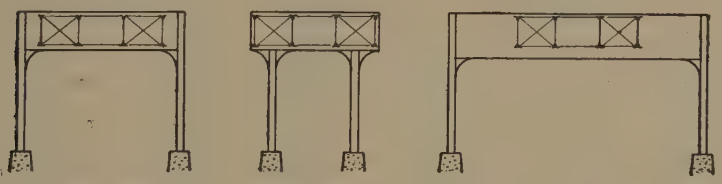


FIG. 98*d*. FIG. 98*e*. FIG. 98*f*.
Types of Girder Bents.

We find three types of supports at bents:

(*I*) Isolated Posts, Unbraced, Fig. 98*b*.
(*II*) Isolated Posts, Braced, Fig. 98*c*.
(*III*) Girder Bents, Fig. 98*d*.

* Engineering Record, Vol. 60, p. 520.

Some economy in the latter could be effected by arranging as shown in Fig. 98*e*. However, a great deal depends on conditions underneath. Possibly, form of Fig. 98*f* may be necessary. If posts must be placed in street, then distance from the curb must be such as to allow a team to pass, say 8 feet in the clear.

There are two types of the longitudinal girder, the plate and the lattice. The former is much like the ordinary type with which we are already familiar; the latter is well explained

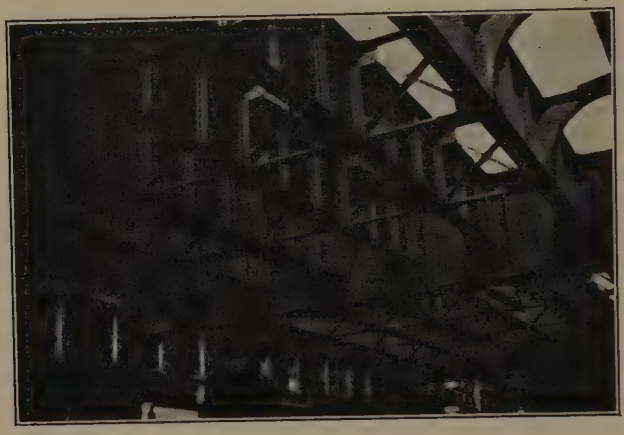


FIG. 98*g*.—Elevated Railroad, Boston, Mass.

by Fig. 98*h*. In either case, avoid cover plates to lessen track labor. Bracing for both is quite similar to that for deck plate girders for ordinary railroads.

Longitudinal girders may rest upon the transverse girders, but "framing in" is stiffer and cheaper for a given clearance. For fixed ends, the plain two-angle connection is employed. At expansion ends, there are various arrangements. One is shown in Fig. 98*i*. Be sure that the driving of field rivets is practicable.

Type (*I*) will have no transverse girders. Type (*II*) should be latticed. Type (*III*) may be either latticed or solid web, each having its advantages, as already explained. Figs. 98*k* and 98*l* show these two types.

Columns

These must be designed for a bending moment in both directions and should offer considerable resistance to collision with vehicles. Unless full bracing is used, they must be fixed at one end; otherwise, they will lack stiffness and might fail.

The best section is the two channel and I, Fig. 56*i*. It should be battened on the sides every three feet. Zee-bar

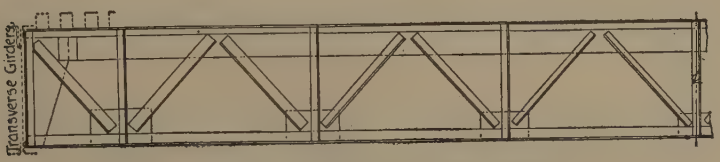


Fig. 98*h*.—Longitudinal Lattice Girder.

columns also make a good design. In New York, a built I with bulb angles is employed. If there is latticing, as in two-channel section, it must be computed to carry the maximum shear in its plane. Neither stiffness nor economy will be promoted by allowing latticing in the plane of any considerable bending. Keep larger resisting moment parallel to the greater bending moment if possible, but do not forget ease and efficiency of connections.

To fix at both ends is stiffer and more economical and the usual arrangement. Fig. 98*m* shows a fixed base for a two-channel and I column. Bolts must be figured to carry maximum bending moment and base must be computed for this as well as the vertical load.

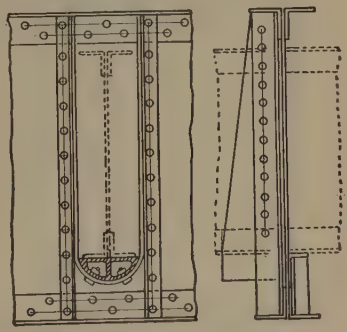


Fig. 98*i*.—" Pocket " Expansion Joint.

As an example of method of computation, let the vertical load for Fig. 98*m* be 100,000 #, and let moment parallel to d_1 be 1,200,000 in. lbs. Let d_1 be 22″; b_1, 30″; b_2, 36″. We may consider that pressure due to moment varies as distance from neutral axis like a beam.

$S = 6M/b_2b_1^2 = 6 \cdot 1{,}200{,}000/36 \cdot 30 \cdot 30 = 222$ lbs. per sq.in.

The vertical load produces $100{,}000/30 \cdot 36 = 93$ lbs. per sq.in.

Maximum compression $= 222 + 93 = 315$ lbs. per sq.in.

Maximum tension $= 222 - 93 = 129$ lbs. per sq.in.

Moment of tensile force is,

$$\frac{129 \cdot 36 \cdot 8.7}{2} \cdot 12.1 = 245{,}000 \text{ inch-lbs.}$$

Stress in each bolt is,

$$245{,}000/2 \cdot 11 = 11100 \text{ lbs.}$$

FIG. 98*j*.—Elevated Railroad Approach to Wells Street. Double Deck Bridge over Chicago River, Chicago, Ill.

It is quite possible that with same moment we might have a smaller vertical load, thus producing a larger stress on bolt. The latter should extend into the masonry and be firmly fastened in such a way that it would lift a weight of masonry equal to the tension before it could be displaced.

Where there are two bending moments at right angles to one another, take their resultant and a moment of resistance about an axis perpendicular to said resultant and treat as above.

To make top fixed, transverse girder should frame into column, not rest upon it, Fig. 98*o*. To further brace bent, we may employ a knee brace, Fig. 98*p*, or a bracket, Fig. 98*o*,

The latter is shown as curved for aesthetic reasons, but the straight bracket is cheaper and more efficient. For Type (*I*), we may use a bracket or flare the column, Fig. 98*q*. The latter is not quite so objectionable as it might seem because where so many are used, a die may be made for bending them. Type (*II*) may be braced transversely by cross frames between girders, Fig. 98*c*, or by latticed struts between the columns, Fig. 98*r*.

Transversely, common practice, which we have just given, provides efficient bracing. Longitudinally, there is divergence

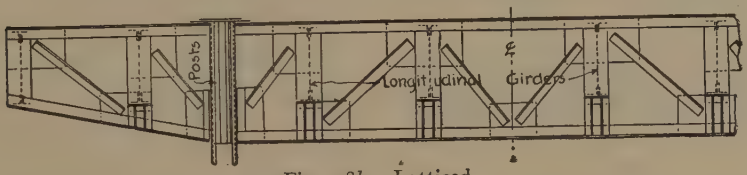


Fig. 98*k*.—Latticed.

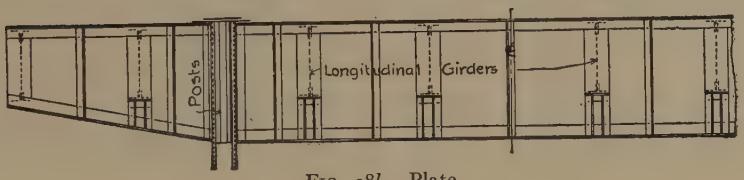


Fig. 98*l*.—Plate.
Transverse Girders.

of opinion, some engineers going so far that they claim that no special provision is necessary for tractive force. However, author would prefer to have value specified below fully taken care of, as shown in Figs. 98*s* and 98*t*. The additional bracing must, of course, be omitted at expansion points.

Solid Floors for Elevated Railroads

From the location of these structures, we should expect solid floors; nevertheless, they are the exception rather than the rule.

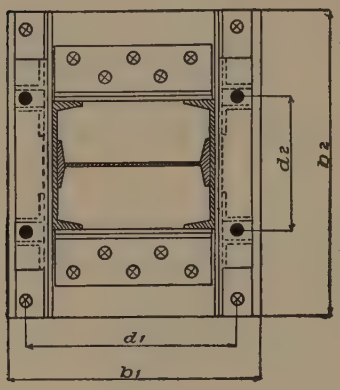
There are two arrangements:

(*A*) Longitudinal troughs resting on floor beams.

(*B*) Transverse troughs supported by deck or through girders.

Loads and Specifications

Loads vary as elevated tracks in city streets may be necessary for anything between a street car and Cooper's E 60.



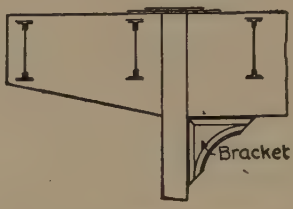


Fig. 98*o*.—Top of Column.

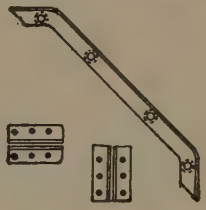


Fig. 98*p*.—Knee Brace.

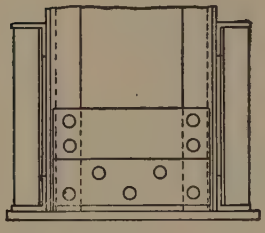


Fig. 98*m*.*—A Fixed Base.

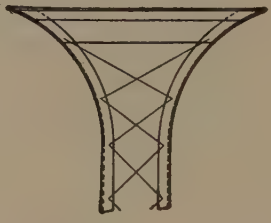


Fig. 98*q*.—Flared Column Top.

Fig. 98*n*.—Distribution of Stress.

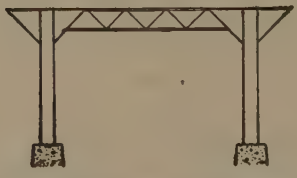


Fig. 98*r*.—Latticed Strut Bracing.

A typical loading per rail is shown in Fig. 98*u*. Centrifugal and wind forces are to be taken as in Art. 50. Longitudinal

* Cavity in base should be filled with concrete to avoid the accumulation of water.

force is to be 10% of total load between expansion points on one track plus 5% on each additional track, distributed over all bents in the same distance. The amount is reduced because the rails and guard timbers help carry the thrust and as a concession to the fact that some engineers make no provision for this force. However, it exists and certainly should be considered in design. Note that even if traffic goes in opposite directions, it is possible that north-bound trains may be applying brakes, while south-bound are just starting.

FIG. 98*s*.—Types (*I*) and (*II*).

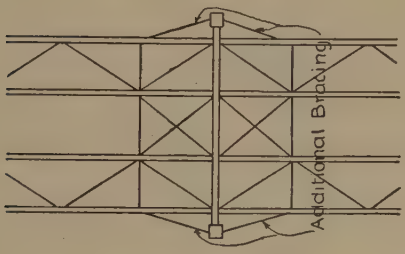


FIG. 98*t*.—Type (*III*).

Bracing for Tractive Forces.

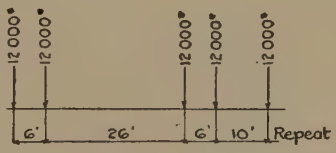


FIG. 98*u*.

Typical Load for an Elevated Railroad.

Remainder of Specifications of Art. 50 may be employed. While loads in elevated railroads are light, traffic is frequent and it is *passenger* traffic. These structures are then not one whit less important than railroad bridges.

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Prob. 98. Design a double track elevated railroad for loads of Fig. 98*s*; clearance underneath, 15 feet; material, medium steel, spans, 50 feet. Columns must be 30 feet apart in the clear.

Art. 99. Computation of Partial Bracing.

Forms of partial bracing are usually fastened at each end. When such a structure is subjected to horizontal or inclined loads, it may be assumed either:

(*1*) That reactions are parallel to resultant load, or

(*2*) That horizontal components of loads are equally divided among the reactions.

(*1*) is very nearly true for a strong and comparatively rigid structure, Fig. 99*a*. (*2*) is approximately correct when portion which carries horizontal load down is relatively weak and elastic, Fig. 99*b*. The latter is the common case in partial bracing and it is easier to handle; hence it is the conventional assumption.

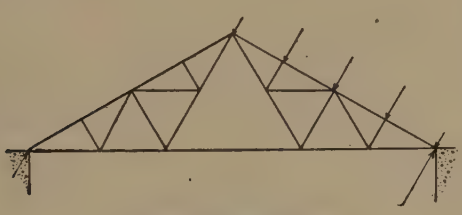


FIG. 99*a*.—Roof Truss.

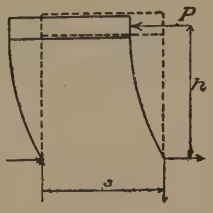


FIG. 99*b*.—Portal.

Partially Braced Structures Fixed at both Ends.

To determine reactions, stresses, and so forth, we proceed in the main as in simple stresses.

The *direct stress* on any transverse section of a member is equal to the *algebraic* sum of *all* components perpendicular to said section which are applied to the member on *either* side of the section.

The *shear* on any transverse section of a member is equal to the *algebraic* sum of *all* components parallel to said section which are applied to the member on *either* side of the section.

The *moment* at any section of a member is equal to the *algebraic* sum of the moments of *all* loads applied to the member on *either* side of the section.

In the above, consider the member as that portion between "hinges." A "theoretic hinge" is a part which can trans-

mit direct stress or shear but not moment. A pin joint, Art. 60, is the best example. The amount of moment which can be carried by an ordinary riveted joint is small and it is usually considered as a hinge.

Where frame work is employed for partial bracing, the ordinary methods of solving by the use of the equations, $\Sigma V = 0$, $\Sigma H = 0$, and $\Sigma M = 0$, are available. It is best, however, not to pass a section through a member carrying moment. While this may be taken into account, it is somewhat difficult, liable to error, and generally unnecessary.

The condition of the structure at the reaction is important.

Where the vertical load under all conditions is sufficient to hold the base down when acted upon by simultaneous transverse forces; or where anchor bolts and all details connected therewith are sufficient for maximum bending moment, the ends are said to be *fixed*.

Where there are short anchor bolts, sufficing only for shear, with small base, just enough for direct stress; or perhaps where an actual pin is used, so placed that it yields under moment, the ends are called *hinged*.

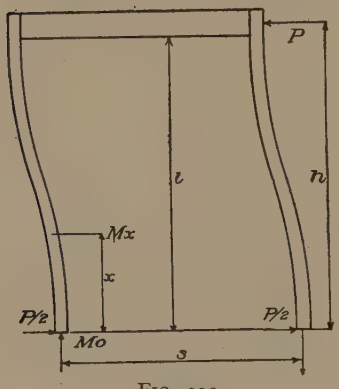


FIG. 99c.
Fixed Ended Partially Braced Bent.

In examining an old structure, try an assumption which seems reasonable. If portions are found to be weak, revise assumptions. Condemn any structure which will not show a proper factor of safety (Art. 68), under any supposition. In designing a new column, we may consider both ends or either one to be fixed and design all parts to fit. However, there may be considerable difference in cost, and great engineering skill is required to select the proper type.

For a structure hinged at the end, Fig. 99*b*, the horizontal component of each reaction will be $P/2$ and the vertical components Ph/s, acting in opposite directions. To determine

either of the latter, we took moments at other reaction. This method cannot be followed in the fixed ended structure seen in Fig. 99*c*.

Let M_0 and Mx be respectively the moments at the point distant o and x from the reactions. The equation of the elastic curve is from Mechanics:

$$EI\frac{\delta^2 y}{\delta x^2}=M_x.$$

But $$M_x=M_0+Px/2.$$

Substituting, $$EI\frac{\delta^2 y}{\delta x^2}=M_0+Px/2.$$

Integrating, $$EI\frac{\delta y}{\delta x}=M_0x+Px^2/4+C_1.$$

When $x=0$, $\delta y/\delta x=0$, hence $C_1=0$.

When $x=l$, $\delta y/\delta x=0$, hence $M_0=-Pl/4$.

Therefore $M_x=P(2x-l)/4$.

When $x=l/2$, $M=0$.

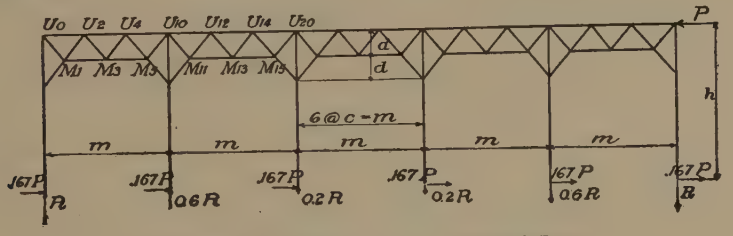


Fig. 99*d*.—Continuous Partially Braced Bent.

Hence we may consider half way point as a joint and find vertical component of reactions to be,

$$P\left(h-\frac{l}{2}\right)/s.$$

We find then that partial bracing fixed at reactions may be regarded as *hinged half way between support* and *bracing*.

All problems hereafter will assume *hinged ends*, the fixed ends being readily converted into the former case.

CONTINUOUS PARTIALLY BRACED BENT. (Fig. 99*d*.)

A series of partially braced bents is considered as follows: We assume, (*A*) horizontal forces equally divided between the different footings; and (*B*), each vertical reaction to vary as its distance from neutral axis as in a beam.

All horizontal components of the reactions are 0.167 *P*. Letting *R* represent the vertical component of the two outermost reactions,

$$2(R\cdot 2.5m+0.6R\cdot 1.5m+0.2R\cdot 0.5m)=Ph.$$

Solving,

$$R=Ph/7m.$$

Let $P=60{,}000$ #, $m=24$ feet, $h=21$ feet, $d=3$ feet.

Then H.C. $=0.167P=10{,}000$ #.

V.C's. $=R$, $0.6R$, and $0.2R=7500$, 4500, and 1500 # respectively. To obtain stress in $U_{12}M_{13}$, we pass a vertical section cutting member, and apply $\Sigma V=0$ to the left. This shows its vertical component to be $7500+4500=12{,}000$ #. Actual stress is $12{,}000\cdot 1.67=20{,}000$ # tension.

To obtain stress in $U_{12}U_{14}$, use same section and apply $\Sigma M=0$, taking moments of forces at left about M_{13}.

$$3\cdot U_{12}U_{14}+7500\cdot 36+4500\cdot 12-20{,}000\cdot 18=0.$$

Solving, $U_{12}U_{14}=12{,}000$ # tension.

Graphically, the stresses in framed part may be obtained by the introduction of assumed members, these being so placed that no member is compelled to carry moment, at the same time causing no change elsewhere. See following problems for examples.

We will take up a few cases to illustrate methods. Members carrying moment will be made heavy. Counters will be represented by dotted lines, assumed members by dots and dashes. Small circles will show hinges. In the graphical solutions, stresses in false members will be shown dotted. We shall consider hinges at reactions in all examples since fixed ends are readily reduced to this condition by changing *h*.

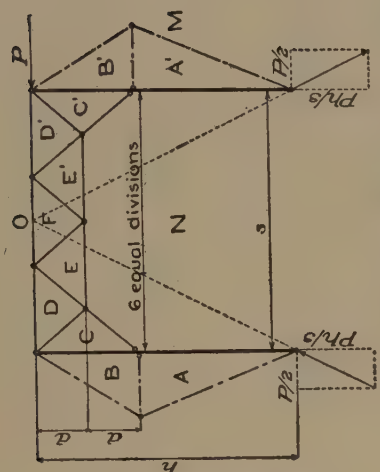


FIG. 99e.—Warren Type. Partially Braced Bent.

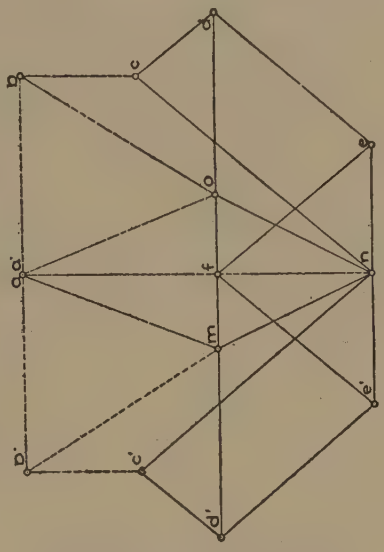


FIG. 99f.—Graphical Solution.

PARTIALLY BRACED BENTS

Case I. Warren Type, with Knee Brace

Member.	Method.	Section Cuts.	Point of Moment.	Stress.	
				Amount.	Nature.
OF	$\Sigma M=0$	*OF, FE, EN*	*EFE'N*	$P/2$	Comp.
OD	$\Sigma M=0$	*OD, DE, EN*	*CDEN*	$P\left(\frac{h}{3d}-\frac{1}{2}\right)$	Tens.
OD'	$\Sigma M=0$	*OD', D'E', E'N*	*D'C'NE'*	$P\left(\frac{h}{3d}+\frac{1}{2}\right)$	Comp.
EN	$\Sigma M=0$	*OF, EF, EN*	*DOFE*	$Ph/6d$	Comp.
E'N	$\Sigma M=0$	*OF, FE', E'N*	*OD'E'F*	$Ph/6d$	Tens.
V.C.EF	$\Sigma V=0$	*OF, FE, EN*	.....	Ph/s	Tens.
V.C.E'F	$\Sigma V=0$	*OF, FE', E'N*	.....	Ph/s	Comp.
V.C.DE	$\Sigma V=0$	*OD, DE, EN*	.....	Ph/s	Comp.
V.C.D'E'	$\Sigma V=0$	*OD', D'E', E'N*	.....	Ph/s	Tens.
V.C.CD	$\Sigma M=0$	*OD, DC, CN*	*DOFE*	$Ph/2s$	Comp.
V.C.C'D'	$\Sigma M=0$	*OD', D'C', C'N*	*FOD'E'*	$Ph/2s$	Tens.
V.C.CN	$\Sigma V=0$	*OD, DC, CN*	.....	$3Ph/2s$	Comp.
V.C.C'N	$\Sigma V=0$	*OD', D'C', C'N*	.....	$3Ph/2s$	Tens.

If we let W represent vertical load applied at each upper outside joint, the maximum stresses in posts are:

Direct $W+Ph/s$, C at the left.

$W+Ph/2s$, C at the right.

Shear $\frac{1}{2}P$.

Max. Mom. $\frac{1}{2}P(h-2d)+Wf$,* where f represents the deflection of the posts, Fig. 99*g*. To obtain its value, let E equal the modulus of elasticity and I the moment of inertia of column

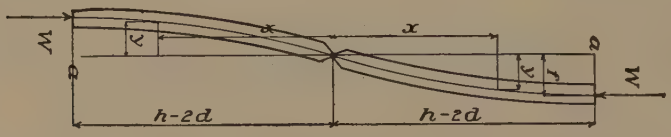


FIG. 99*g*.—Fixed Post under Direct and Transverse Loads.

about an axis perpendicular to P. Proceeding as in (2) of Art. 55.

Deflection due to a bending moment $m=Pl$ is $Pl^3/3EI = ml^2/3EI$.

Assuming now that the deflection f bears a similar ratio to its moment, M:

$$M=\tfrac{1}{2}P(h-2d)+WM(h-2d)^2/3EI,$$

whence

$$M=\frac{P}{2\left(\frac{1}{h-2d}-\frac{W(h-2d)}{3EI}\right)}.$$

* Many engineers ignore the term Wf. It is of little importance in short stiff columns, and may generally be omitted.

Case II. Abbreviated Warren Type with Knee Brace

This might also be regarded as a special problem arising under the preceding case. The bent is seen in Fig. 99*h*, while Fig. 99*i* shows the graphical solution. By the analytical method, we first find stress in *EN* by passing a section through *OD*, *DE*, and *EN*, and taking moments about *OD'ED*. We obtain zero. $\Sigma V = 0$ at *CDEN* with *H* axis in *DE* gives

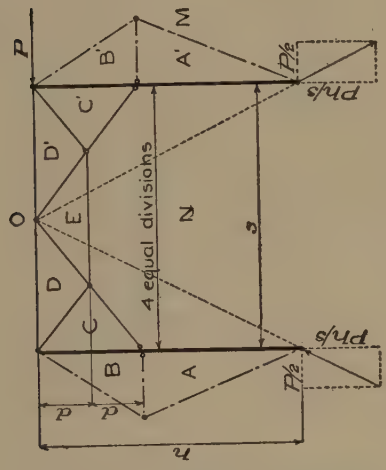


Fig. 99*h*.—Abbreviated Knee-braced Warren Truss Bent.

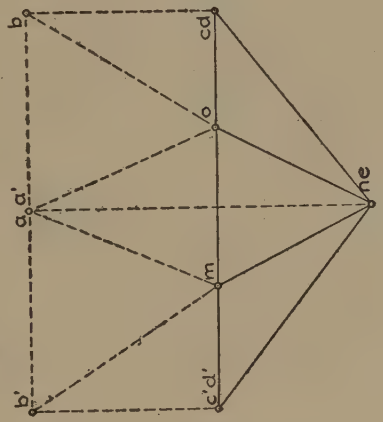


Fig. 99*i*.—Graphical Solution.

$CD = 0$. Likewise we may obtain $C'D' = 0$. *CN* and *DE* are now seen to be equal. By passing a section through either of these and *DO*, we find that it is in compression, having a vertical component of Ph/s; similarly *D'E* and *C'N* may be found to have a like amount in tension. Using same sections and taking moments about any point in the diagonal, we obtain

$P\left(\frac{h}{4d} - \frac{1}{2}\right)$ tension for *OD* and $P\left(\frac{h}{4d} + \frac{1}{2}\right)$ compression for *OD'*.

Values for post are as given in preceding case.

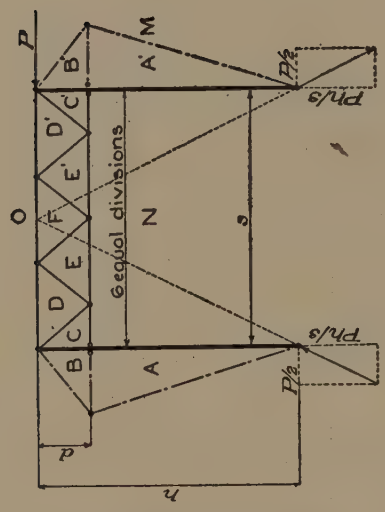


FIG. 99*j*.—Bent Partially Braced by Plain Warren Truss.

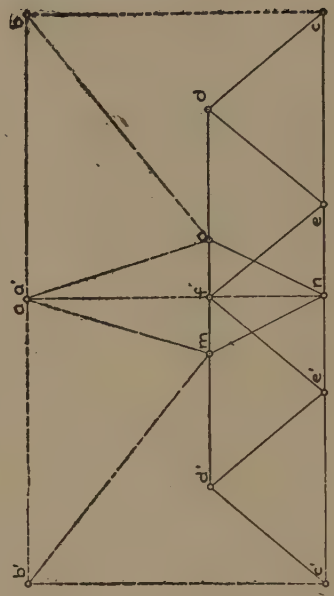


FIG. 99*k*.—Graphical Solution.

Case III. Warren Truss, Plain

Member.	Method.	Section Cuts.	Point of Moment.	Stress.	
				Amount.	Nature.
OF	$\Sigma M = 0$	*OF, EF, EN*	*EFE'N*	$0.5P$	Comp.
OD	$\Sigma M = 0$	*OD, CD, CN*	*CDEN*	$P\left(\frac{h}{3d} - \frac{1}{2}\right)$	Tens.
OD'	$\Sigma M = 0$	*OD', C'D', C'N*	*E'D'C'N'*	$P\left(\frac{h}{3d} + \frac{1}{2}\right)$	Comp.
CN	$\Sigma M = 0$	*OD, DC, CN*	*ODCB*	$Ph/2d$	Comp.
C'N	$\Sigma M = 0$	*OD', D'C', C'N*	*OD'C'B'*	$Ph/2d$	Tens.
EN	$\Sigma M = 0$	*OF, FE, EN*	*OFED*	$Ph/6d$	Comp.
E'N	$\Sigma M = 0$	*OF, FE', E'N*	*FOD'E'*	$Ph/6d$	Tens.
V.C.CD	$\Sigma V = 0$	*OD, CD, CN*	.....	Ph/s	Tens.
V. C. C'D'	$\Sigma V = 0$	*OD', C'D', C'N*	.....	Ph/s	Comp.
V.C.DE	$\Sigma V = 0$	*OD, DE, EN*	.....	Ph/s	Comp.
V.C.D'E'	$\Sigma V = 0$	*OD', D'E', E'N*	.....	Ph/s	Tens.
V.C.EF	$\Sigma V = 0$	*OF, FE, EN*	.....	Ph/s	Tens.
V.C.E'F	$\Sigma V = 0$	*OF, FE', E'N*	.....	Ph/s	Comp.

$$\text{Posts}\begin{cases}\text{Maximum direct stress, } Ph/s. \text{ Compression on left, tension on right;}\\ \text{Maximum shear, } \tfrac{1}{2}P \text{ or } \dfrac{P}{2}\left(1-\dfrac{h}{d}\right);\\ \text{Maximum moment } \dfrac{P}{2\left(\dfrac{1}{h-d}-\dfrac{W(h-d)}{3EI}\right)},\end{cases}$$

the latter term in denominator being often omitted.

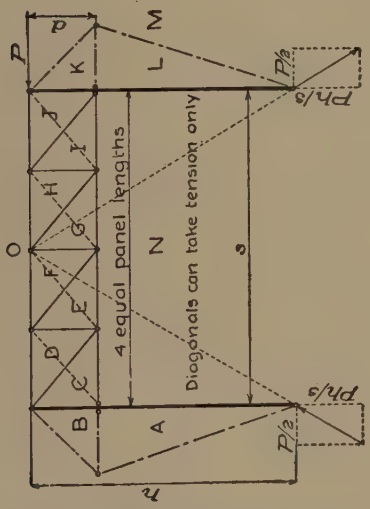


FIG. 99*l*.—Bent Partially Braced by Plain Pratt Truss.

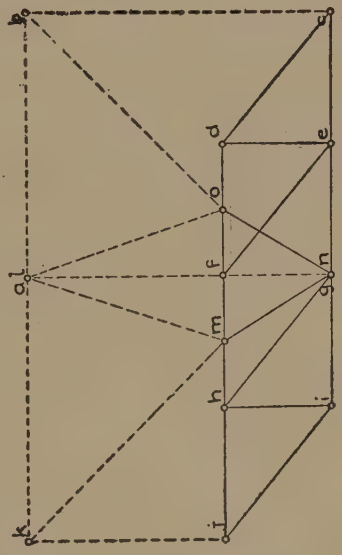


FIG. 99*m*.—Graphic Solution.

Case IV. Pratt Truss, Plain

By passing a section through upper and lower chord and also the web member whose stress is required, it is found that all vertical components are Ph/s. This is compression for vertical members and must of course be tension for diagonals. By method of moments, remaining stresses may be ascertained as follows:

Member.	Stress.		Member.	Stress.	
	Amount.	Nature.		Amount.	Nature.
OD	$P\left(\frac{h}{4d}-\frac{1}{2}\right)$	Tens.	*CN*	$\frac{Ph}{2d}$	Comp.
OF	$0.5P$	Comp.	*EN*	$Ph/4d$	Comp.
OH	$P\left(\frac{h}{4d}+\frac{1}{2}\right)$	Comp.	*GN*	0	
OJ	$P\left(\frac{h}{2d}+\frac{1}{2}\right)$	Comp.	*IN*	$Ph/4d$	Tens.

Posts:

Maximum direct stress Ph/s, compression on left, tension on right.

Maximum shear, $0.5P$ or $\frac{Ph}{2d}-\frac{P}{2}$.

Maximum moment, $\frac{P}{2\left(\frac{1}{h-d}-W\frac{h-d}{3EI}\right)}$.

Case V. Knee-braced Bent

Since there can be no moments at the upper corners, we readily obtain that the vertical components of the stresses in knee braces are—

$Ph/2c$ Comp. and $Ph/2c$ Tens.

Corresponding horizontal components are $Ph/2d$. We may now compute moments and plot moment diagrams as given in Fig. 99*n*.

Case VI. Girder-braced Bent

Due to horizontal loads only, the bending moment at any point y is $Phy/s-Ph/2$; at any point x, it is $Px/2$. The deflection may be taken into account in the manner indicated under Case I.

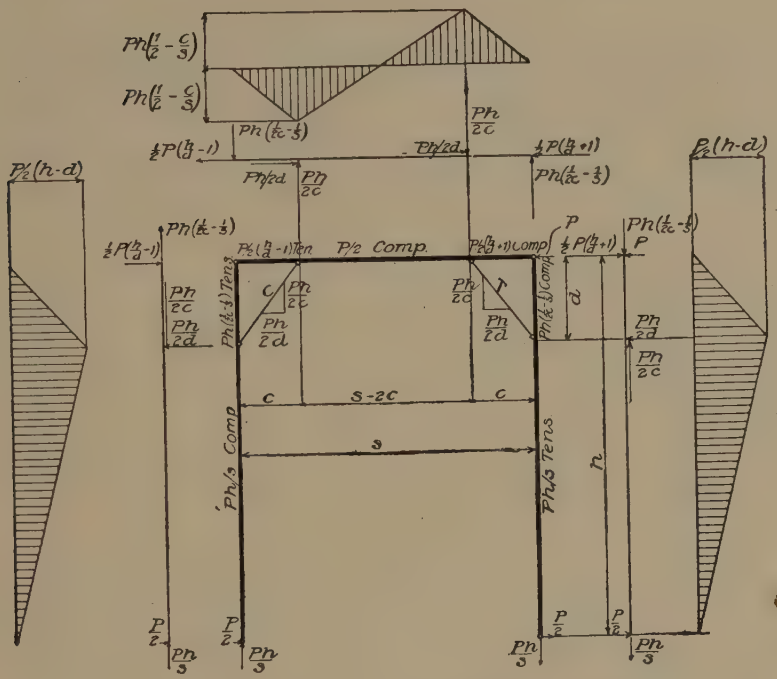


FIG. 99*n*.—Kneebraced Bent, with Moment Diagrams.

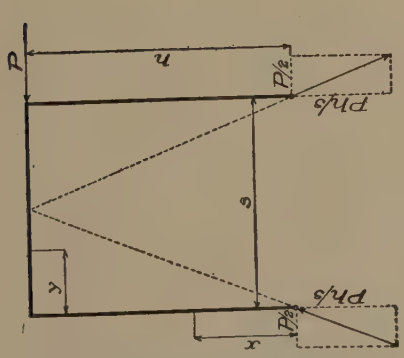


FIG. 99*o*.—Girder-braced Bent.

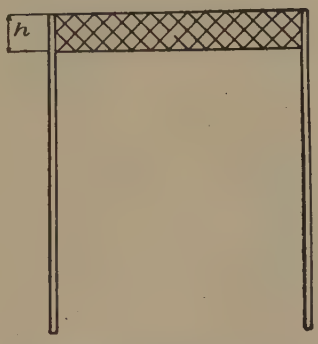


FIG. 99*p*.—Latticed Portal.

Case VII. Lattice-braced Bent

A latticed portal, Fig. 99*p*, may be treated as a case of this kind. The shear, Ph/s, will be distributed equally among the vertical components of all the diagonals cut. The moment at any point in horizontal members may be assumed to be carried entirely by the chords, and, as in the approximate method for plate girders, Art. 54,

$$a = M/Sh,$$

where a = required area;
M = bending moment;
h = distance between centers of gravity of flanges;
S = allowable unit stress.
Everything in pounds and inches.

REFERENCE

"Design of Highway Bridges," by Ketchum, p. 112.

Prob. 99. In Fig. 99*p*, let the total height be 20 feet; center to center posts, 12 feet; and depth h 4 feet. The single load of 20,000 lbs. acts horizontally at top on either side in either direction. In addition, there is a direct stress of 164,000 lbs. Considering as pin ended at bottom, design in soft steel for a highway bridge.

Art. 100. Computation of a Steel Viaduct

Let it be required to design a steel viaduct for a single track on a tangent. Height from base of rail to maximum high water, 100 feet. See Fig. 100*c* for profile. Loads and specifications as in Art. 50 except that live load is 5000 pounds per lineal foot plus two concentrations 50 feet apart of 50,000 pounds each, all per track, and that we shall ignore stresses due to own weight in wind bracing of plate girders. Use medium steel except for rivets, which are $\frac{7}{8}$ inch in the main and of soft steel.

We will use tower spans of 40 feet; intermediate spans of 80 feet; girders, 7′ center to center; bents at top, the same; bevel of legs, 2″ in 12″; diagonals, stiff tension members; computations for girders by the intermediate method, Art. 54; stresses due to own weight determined by the first method of Art. 55.

Girders

Economical Depth. Taking 3200 lbs. per lin.ft. as load per girder, and 11,000 lbs. per sq.in. as average allowable unit stress, we have that their economical depth is,

$$h = 1.75\sqrt{M/S} = 93'' \text{ depth for 80-foot lengths;}$$
$$47'' \text{ depth for 40-foot lengths.}$$

$$4 \times 93 = 372$$
$$1 \times 47 = 47$$
$$5 \times 84 = 419$$

Since total weight varies as square of the length, we must attach four times as much weight to the longer span.

We will employ 84″ plate for both with 7′ 0½″ back to back angles.

Estimated Weight

$$W = 18.0\, l^2\sqrt{w/S} = 18 \cdot 80 \cdot 80\sqrt{6400/11{,}000} = 88{,}000 \text{ lbs. for 80-foot span;}$$
$$= 18 \cdot 40 \cdot 40\sqrt{6400/11{,}000} = 22{,}000 \text{ lbs. for 40-foot span.}$$

Both are for entire bridge.

Shears and Moments per Girder

Shears in Kips * Moments in Kip Feet

40-Foot Span, Shears

Loads.	Reac.	5′ out.	10′ out.	15′ out.	Mid.
	+	+	+	+	
Dead, 475 lbs. per lin.ft.	9.5	7.2	4.8	2.4	0.0
	+	+	+	+	+
	50.0	38.3	28.1	19.5	12.5
Live, 2500 lbs. per lin.ft.		−	−	−	−
		0.8	3.1	7.0	12.5
	+	+	+	+	+
	25.0	21.9	18.8	15.6	12.5
Concentrated, 25,000 lbs.		−	−	−	−
		3.1	6.3	9.4	12.5
	+	+	+	+	+
Maximum	84.5	67.4	51.7	37.5	25.0
	+	+	−	−	−
Minimum	9.5	3.3	4.6	14.0	25.0
$1+\frac{1}{2}\frac{\text{Minimum}}{\text{Maximum}}$	1.056	1.024	0.956	0.813	0.500

* 1 kip = 1000 lbs.

80-Foot Span, Shears

Loads.	Reac.	10′ out.	20′ out.	30′ out.	Mid.
	+	+	+	+	
Dead, 750 lbs. per lin.ft.	30.0	22.5	15.0	7.5	0.0
	+	+	+	+	+
Live, 2500 lbs. per lin.ft.	100.0	76.5	56.2	39.0	25.0
		−	−	−	−
		1.5	6.2	14.0	25.0
	+	+	+	+	+
Conc. 25,000 lbs. each.	34.4	28.1	21.9	15.6	12.5
	−	−	−	−	−
		3.1	6.3	9.4	12.5
	+	+	+	+	+
Maximum.	164.4	127.1	93.1	62.1	37.5
	+	+	+	−	−
Minimum.	30.0	17.9	2.5	15.9	37.5
$1+\frac{1}{2}\frac{\text{Minimum}}{\text{Maximum}}$.	1.091	1.070	1.013	0.871	0.500

Moments and Flanges, 40-Foot Spans

	5′ out.	10′ out.	15′ out.	Mid.
	+	+	+	+
Dead Load, 475 lbs. per lin.ft.	42	71	89	95
	+	+	+	+
Live Load, 2500 lbs per lin.ft.	219	375	469	500
	+	+	+	+
Concentrated Load, 25,000 lbs.	109	188	234	250
	+	+	+	+
Maximum.	370	634	792	845
	+	+	+	+
Minimum.	42	71	89	95
$1+\frac{1}{2}\frac{\text{Minimum}}{\text{Maximum}}$.	1.056	1.056	1.056	1.056
Flange stress $=\frac{\text{Moment}}{h=6.8'}$.	54.3	93.0	116.2	124.0
$\frac{\text{Flange Stress}}{\text{Allowable Unit Stress}}=\left(\begin{matrix}\text{Area reqd.}\\ \text{in sq.in.}\end{matrix}\right)$.	5.10	8.80	11.00	11.75
Deduct equivalent area web as given below $=\frac{1}{8}\cdot\frac{3}{8}\cdot 84=4.0$.	1.10	4.80	7.00	7.75

Moments and Flanges, 80-foot Span

	10′ out.	20′ out.	30′ out.	Mid.
	+	+	+	+
Dead Load, 750 lbs. per lin.ft.	262	450	563	600
	+	+	+	+
Live Load, 2500 lbs. per lin.ft.	875	1500	1875	2000
	+	+	+	+
Concentrated Loads, 25,000 lbs.	281	438	469	500
	+	+	+	+
Maximum	1418	2388	2907	3100
	+	+	+	+
Minimum	262	450	563	600
$1+\frac{1}{2}\frac{\text{Minimum}}{\text{Maximum}}$	1.093	1.094	1.097	1.097
$\text{Flange Stress}=\frac{\text{Moment}}{h=6.9'}$	205	346	421	449
$\frac{\text{Flange Stress}}{\text{Allowable Unit Stress}}=\text{Area reqd. in sq.in.}$	18.7	31.6	38.4	40.9
Deduct equivalent area web as given below $=\frac{1}{8}\cdot\frac{3}{8}\cdot 84=4.0$ sq. in.	14.7	27.6	34.4	36.9

Thickness of Web

$$\text{40-foot span, } t=\frac{84{,}500}{6700\cdot 1.056\cdot 84}=.143'', \quad \text{use } \tfrac{3}{8}''.$$

$$\text{80-foot span, } t=\frac{164{,}400}{6700\cdot 1.091\cdot 84}=.268'', \quad \text{use } \tfrac{3}{8}''.$$

Flanges

For the 40-foot span, we will use throughout 2 L's, $6''\times 6''\times\frac{3}{8}''$; gross area, 8.72; net, 7.98 sq.ins. In spite of the fact that the angles alone furnish sufficient strength, it is probably best to employ a $14''\times\frac{3}{8}''$ cover plate on top to protect from weather. For 80-foot length, we will use a flange of type *B*, Fig. 79*a*, for illustrative purposes rather than for economy. We may now plot flange diagram in Fig. 100*c*.

Rivet Spacing

From preceding problems, it will be obvious that the maximum allowable spacing will do for vertical rivets. The term "Fraction" is used to designate that decimal part of flange to which stresses are transmitted by rivets under consideration. Two computations are necessary at each point, for 80-foot girder; one for rivets between side plates and web and another for those between side plates and angles. We have considered $1+\frac{1}{2}\frac{\text{Min.}}{\text{Max.}}$ to equal 1.00 for all cases; this is only roughly correct. In table below, all values are in pounds and inches and relate to rivets in the upper flange. Spacing in the lower chord should be made the same.

Spacing, 40-Foot Span

Dist. from Reac. feet.	V/h per Lin.in.	Fraction.	Shear per Lin.in.	Vert. Force per Lin.in.	Resultant	$\frac{7}{8}$" Rivet.	Value.	Spacing
0	1030	0.76	780	900	1190	Bg. on $\frac{3}{8}$"	3940	3.30
5	820	0.76	620	900	1090	Bg. on $\frac{3}{8}$"	3940	3.60
10	630	0.76	480	900	1020	Bg. on $\frac{3}{8}$"	3940	3.80
15	460	0.76	350	900	970	Bg. on $\frac{3}{8}$"	3940	4.10
20	310	0.76	240	900	930	Bg. on $\frac{3}{8}$"	3940	4.20

80-Foot Span

0	1980	0.86	1700	900	1920	Bg. on $\frac{3}{8}$"	3940	2.05
		0.52	1030	900	1370	Dou. Sh.	7200	5.30
10	1530	0.86	1320	900	1600	Bg. on $\frac{3}{8}$"	3940	2.45
		0.52	800	900	1200	Dou. Sh.	7200	6.00
20	1120	0.89	1000	900	1350	Bg. on $\frac{3}{8}$"	3940	2.90
		0.60	680	900	1130	Dou. Sh.	7200	6.40
30	750	0.90	680	900	1130	Bg. on $\frac{3}{8}$"	3940	3.50
		0.66	500	900	1030	Dou. Sh.	7200	7.00
40	450	0.90	410	900	990	Bg. on $\frac{3}{8}$"	3940	4.00
		0.66	300	900	950	Dou. Sh.	7200	7.60

If the rivets are placed two in a row,—they probably will be between side plates and web,—spacing should be *doubled*.

Pairs of $5'' \times 3\frac{1}{2}''$ L's will be used as stiffeners; on the 40-foot span, they will be $1\frac{1}{8}''$ apart; for the 80-foot, $2\frac{1}{8}''$; in the former case, the radius of gyration and allowable stress are 2.70″ and 8600 lbs. per sq.in.; in the latter, 3.10 and 9200.

For 40-foot span, 84,500/8600 = 9.83 sq.in.

Use 4 L's, $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, area 12.20 sq.in.

For 80-foot span, 164,400/9200 = 17.90 sq.in.

Use 4 L's, $5'' \times 3\frac{1}{2}'' \times \frac{9}{16}''$, area 17.88 sq.in.

The above are the end stiffeners; the intermediate for both spans will be made $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$.

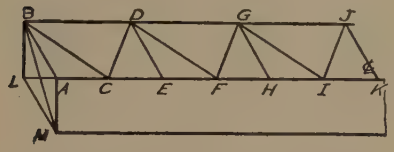


Fig. 100*a*.
Diagram of Wind Bracing for Half of 80-foot Span.

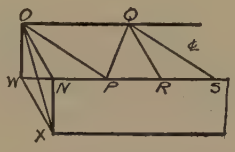


Fig. 100*b*.
Diagram of Wind Bracing for Half of 40-foot Span.

Wind Bracing

The live wind of 30 lbs. per sq.ft. on 10 vertical feet of train makes 300 lbs. per lin.ft., which acts as horizontal and vertical forces on rail. The latter will be found to be without influence on design. The former we will take in conjunction with a dead wind of $30 \cdot 8/2 = 120$ lbs. per lin.ft., as acting on top chord. Allowing 80% of the load, Art. 50, *c*2, panel loads become:

Top, dead, 0.65 and 1.30 kips; live, 1.6 and 3.2 kips.
Bottom, dead, 0.65 and 1.3 kips.

Value of field rivets in single shear is, $0.6 \cdot 4500\left(1 + \frac{1}{2}\frac{\text{Min.}}{\text{Max.}}\right)$.

Smallest allowable number of rivets in a diagonal, 3; for other members, 2. Bottom laterals, which are not computed, will be made like the corresponding top laterals.

Everything in kips and inches.

Member.	Maximum.	Minimum.	Unsupported Length.	Use 1 L	ρ	Min. / Max.	Allow. Unit Stress.	Area.	Allowable Total	Field Rivets Sin. Sh.
OP	7.7C	6.2T	116	6″×6″×⅜″	1.19	0.597	2480	4.36	10.8	5
PQ	6.2C	5.1T	116	6″×6″×⅜″	1.19	0.590	2450	4.36	10.7	4
QS	2.7C	1.5T	116	6″×6″×⅜″	1.19	0.722	3000	4.36	13.1	3
QR	2.3C	0	84	4″×4″×⅜″	0.79	1.000	3640	2.86	10.3	2
ON	2.9C	2.6T	84	4″×4″×⅜″	0.79	0.552	2010	2.86	5.7	2
NW, *OX*	4.8C	4.8T	60	4″×4″×⅜″	0.79	0.500	2700	2.86	7.7	4
BC	17.1C	15.6T	116	6″×6″×11/16″	1.17	0.544	2200	7.78	17.1	12
CD	15.6C	14.2T	116	6″×6″×⅝″	1.18	0.544	2230	7.11	15.8	11
DF	11.5C	10.1T	116	6″×6″×7/16″	1.19	0.560	2330	5.06	11.8	8
FG	10.1C	8.8T	116	6″×6″×⅜″	1.19	0.565	2350	4.36	10.3	7
GI	6.5C	5.3T	116	6″×6″×⅜″	1.19	0.592	2460	4.36	10.7	4
IJ	5.3C	4.3T	116	6″×6″×⅜″	1.19	0.595	2470	4.36	10.7	4
DE, *GH*, *JK*	2.3C	0	84	4″×4″×½″	0.78	1.000	3550	3.75	13.3	2
BA	6.7C	6.0T	84	4″×4″×½″	0.78	0.553	2010	3.75	7.5	5
AL, *BM*	9.6C	9.6T	60	4″×4″×½″	0.78	0.500	2700	3.75	10.1	7

Loads

A rough computation now shows the weight of the bridges to be about 28,000 and 86,000 lbs. for short and long spans respectively, instead of 22,000 and 88,000 as estimated. The former is in excess because it is above its own economical depth. However, we will continue to use original values.

We now have,

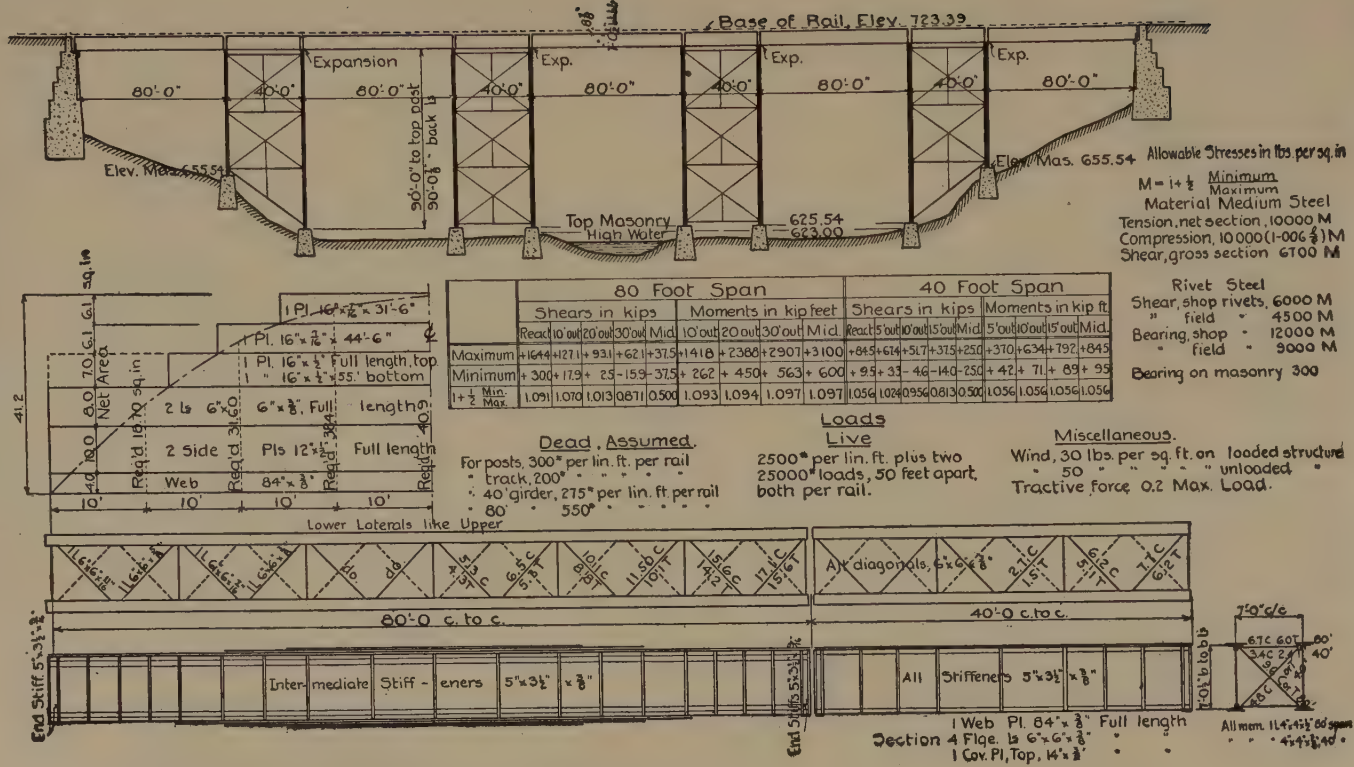
Estimated weight of posts and bracing 300 lbs. per lin.ft.

Dead load applied vertically at the top of each post,

$$475 \cdot 20 + 750 \cdot 40 + 300 \cdot 15 = 44{,}000 \text{ lbs.}$$

Live load applied vertically at the top of each post,

$$2500 \cdot \frac{40+80}{2} + 25{,}000(1+30/80) = 184{,}000 \text{ lbs.}$$


Base of Rail, Elev. 723.39
Expansion
Exp.
Exp.
Exp.
80'-0"
40'-0"
80'-0"
40'-0"
80'-0"
40'-0"
80'-0"
90'-0" to top post
90'-0" - back to back
Elev. Mas. 655.54
Elev. Mas. 655.54
Top Masonry
High Water
625.54
623.00
Allowable Stresses in lbs. per sq. in
M = 1 + 1/2 Minimum / Maximum
Material Medium Steel
Tension, net section, 10000 M
Compression, 10000 (1 - 0.006 l/r) M
Shear, gross section 6700 M
Rivet Steel
Shear, shop rivets, 6000 M
" field " 4500 M
Bearing, shop " 12000 M
" field " 9000 M
Bearing on masonry 300
1 Pl. 16" x 31'-6"
1 Pl. 16" x 44'-6"
1 Pl. 16" x Full length top
1 " 16" x 55.' bottom
2 L 6" x 6" x Full length
2 Side Pls 12" x Full length
Web 84" x
Net Area
Lower Laterals like Upper
80'-0 c. to c.
40'-0 c. to c.
All diagonals, 6" x 6"
All Stiffeners 5" x 3 1/2" x
End Stiffs 5" x 3 1/2" x
Inter-mediate Stiff-eners 5" x 3 1/2" x
1 Web Pl. 84" x Full length
Section 4 Flge. L 6" x 6" x
1 Cov. Pl, Top, 14" x
7'-0" c/c
All mem. 11.4 - 11.4 - 11.4 - 80 span
" " 4 - 4 - 4 - 40 "

	80 Foot Span									40 Foot Span								
	Shears in kips					Moments in kip feet				Shears in kips					Moments in kip ft.			
	React	10' out	20' out	30' out	Mid.	10' out	20' out	30' out	Mid.	React	5' out	10' out	15' out	Mid.	5' out	10' out	15' out	Mid.
Maximum	+164.4	+127.1	+93.1	+62.1	+37.5	+141.8	+238.8	+290.7	+310.0	+84.5	+61.4	+51.7	+37.5	+25.0	+37.0	+63.4	+79.2	+84.5
Minimum	+30.0	+17.9	+2.5	-15.9	-37.5	+26.2	+45.0	+56.3	+60.0	+9.5	+3.3	-4.6	-14.0	-25.0	+4.2	+7.1	+8.9	+9.5
1 + 1/2 Min. / Max	1.091	1.070	1.013	0.871	0.500	1.093	1.094	1.097	1.097	1.056	1.024	0.956	0.813	0.500	1.056	1.056	1.056	1.056

Loads

Dead, Assumed.
For posts, 300# per lin. ft. per rail
" track, 200# " " " "
" 40' girder, 275# per lin. ft. per rail
" 80' " 550# " " " "

Live
2500# per lin. ft. plus two 25000# loads, 50 feet apart, both per rail.

Miscellaneous.
Wind, 30 lbs. per sq. ft. on loaded structure
" 50 " " " " " unloaded "
Tractive force 0.2 Max. Load.

FIG. 100c.—Strain Sheet of Girders and Profile.

Dead load applied vertically at each intermediate joint,

$$30 \cdot 300 = 9000 \text{ lbs.}$$

Wind load applied perpendicular to the girder, either,

$$30 \cdot 8 \cdot \frac{80+40}{2} = 14{,}400 \text{ lbs. 4 ft. above column,}$$

together with,

$$30 \cdot 10 \cdot \frac{80+40}{2} = 18{,}000 \text{ lbs. 17 ft. above column.}$$

This is to be on loaded structure; or, applied in the same manner,

$$50 \cdot 8 \cdot \frac{80+40}{2} = 24{,}000 \text{ lbs., 4 ft. above column,}$$

on bridge without live load.

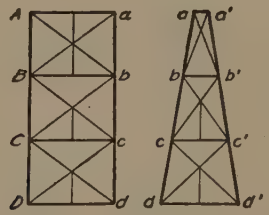


FIG. 100*d*.—Outline of Towers.

Tractive force applied parallel to track at base of rails, is for each side of bent,
$0.2[2500(40+80)+2 \cdot 25{,}000] = 70{,}000$ lbs., 8 ft. above column.

COMPUTATION OF STRESSES, FIG. 100*d*.

We may now compute stresses assuming that the towers will be divided into stories just 30 feet high, although they will be detailed with a space both top and bottom and an interval between struts of about 29 feet, see Fig. 100*f*. Tabulation of stresses follows:

Loads	Posts.			Trans. Diags.			Trans. Struts.				Long. Diags.			Long. Struts.			
	ab	*bc*	*cd*	*ab′*	*bc′*	*cd′*	*aa′*	*bb′*	*cc′*	*dd′*	*Ab*	*Bc*	*Cd*	*Aa*	*Bb*	*Cc*	*Dd*
Dead	C 44.6	C 53.7	C 62.9	....	...	...	C 7.3	C 1.5	C 1.5	T 10.3							
Live	C 186.6	C 186.6	C 186.6	....	...	...	C 30.7	...	...	T 30.7							
Wind, 30 #	C 79.7 / T 52.7	C 86.7 / T 79.7	C 90.0 / T 86.7	T 28.6	T 8.5	T 4.6	C 7.5	C 6.2	C 3.9	C 1.4							
Wind, 50 #	C 48.8 / T 14.2	C 57.8 / T 48.8	C 61.9 / T 57.8	T 36.7	T 11.0	T 5.8	C 9.7	C 8.0	C 5.0	C 1.8							
Brakes	C 66.5 / T 14.0	C 119.0 / T 66.5	C 171.5 / T 119.0	..	...	...	...	...	...	...	T 87.5	T 87.5	T 87.5	C 35.0	C 70.0	C 70.0	C 35.0
Maximum *	C 301.5	C 356.8	C 408.8	T 29.4	T 8.8	T 4.5	C 38.0	C 7.6	C 5.2	T 41.0	T 70.0	T 70.0	T 70.0	C 28.0	C 56.0	C 56.0	C 28.0
Minimum *	C 24.3	C 4.0	C 4.1	0	0	0	C 7.3	C 1.5	C 1.5	T 8.5	0	0	0	0	0	0	0
$1+\frac{1}{2}\frac{\text{Min}}{\text{Max}}$	1.040	1.006	1.005	1.000	1.000	1.000	1.096	1.100	1.145	1.103	1.000	1.000	1.000	1.000	1.000	1.000	1.000

* Reduced 20% where wind is included, see Art. 50, (*c*2).

Design of Posts

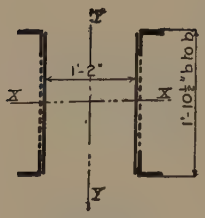


Fig. 100*e*.
Section of Post.

Everything in lbs. and inches.
Try

	Area.	I_{xx}.	I_{yy}
2 Pls. $22'' \times \frac{9}{16}''$	24.75	998	1312
4 L's, $4'' \times 4'' \times \frac{1}{2}''$	15.00	1543	1168
Totals	39.75	2541	2480

$$\rho = \sqrt{2480/39.75} = 7.90''.$$

Capacity for *ab* is $39.75 \cdot 10{,}400\left(1 - .006 \cdot \frac{360}{7.90}\right) = 301{,}000$ lbs.

O.K.

For next length we add side plates with results as follows:

2 Pls. $14'' \times \frac{3}{8}''$	10.50	172	631
Totals	50.25	2713	3111

$$\rho = \sqrt{2713/50.25} = 7.34''.$$

Capacity for *bc* is $50.25 \cdot 10{,}060\left(1 - .006 \cdot \frac{360}{7.34}\right) = 357{,}000$ lbs.

O.K.

For the lower length, we change thickness of angles from $\frac{1}{2}''$ to $\frac{3}{4}''$.

4 L's, $4'' \times 4'' \times \frac{3}{4}''$	21.76	2188	1729
Totals	57.01	3358	3672

$$\rho = \sqrt{3358/57.01} = 7.65''.$$

Capacity for *cd* is $57.01 \cdot 10{,}050\left(1 - .006 \cdot \frac{360}{7.65}\right) = 411{,}000$ lbs.

O.K.

These sections were selected after several trials. Not only must they be just strong enough without any surplus, but sections should splice readily.

Design of Diagonals

	ab'.	*bc'*	*cd'*.	Longitudinal.
Maximum tension, kips	29.4	8.8	4.5	70.0
$1+\frac{1}{2}\frac{\text{Minimum}}{\text{Maximum}}$	1.000	1.000	1.000	1.000
Try two angles		$3\frac{1}{2}''\times 3\frac{1}{2}''\times\frac{3}{8}''$		($6''\times 4''\times\frac{5}{8}''$, 6″ leg vertical)
Net area, sq.in.	4.22	4.22	4.22	10.48
Direct unit stress, lbs. per sq.in.	7,000	2,100	1,100	6,700
Max. unsupported length, feet	22.8	22.8	25.3	25.0
Horizontal projection in feet	8.5	13.5	18.5	20.0
Estimated weight in lbs. per lin.ft.	20	20	20	45
Flexural stress, lbs. per sq.in.*	2,500	4,000	6,100	3,200
Total stress, lbs. per sq. in.	9,500	6,100	7,200	9,900
Allowable stress, lbs. per sq..in.	10,000	10,000	10,000	10,000

Design of Struts

	aa'.	*bb'*.	*cc'*.	*dd'*.	Longitudinals.
Max. stress, kips	38.0C	7.6C	5.2C	41.0T	56.0C
$1+\frac{1}{2}\frac{\text{Minimum}}{\text{Maximum}}$	1.096	1.100	1.145	1.103	1.000
Try two channels	8″	@	16.25#,		10″@20#
Gross area, sq.in.	9.56	9.56	9.56	9.56	11.76
Net area, sq.in.	....	....	....	7.96	
Direct stress, lbs. per sq.in.	4000	800	550	5150	4750
Channels turned	Out	Out	Out	Out	In
Back to back, ins.	14	14	14	14	21.5
Radii of gyration in ins., vert.	2.89	2.89	2.89	.....	3.66
Radii of gyration in ins., hor.	7.6	7.6	7.6	.....	10.1
Unsupported length, inches, vert.	84	204	162	222	240
Unsupported length, inches, hor.	84	204	324	....	480
Estimated weight lbs. per lin.ft.	50	50	50	50	60
Flexural stress, lbs. per sq. in.	180	1080	680	1280	1150
Total stress, lbs. per sq.in.	4180	1880	1230	6430	5900
Allowable stress, lbs. per sq.in.	9000	6320	7650	11000	6070

* Treated as inclined members, Art. 55.

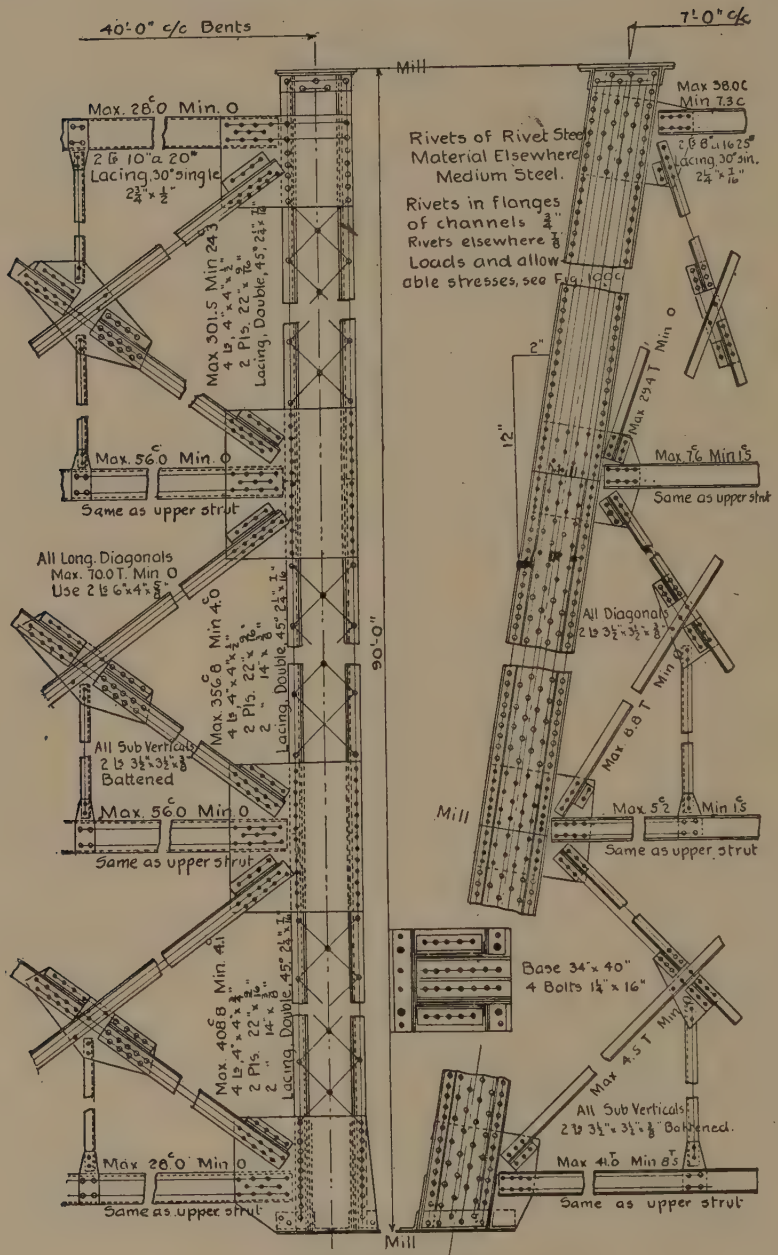


FIG. 100f.—General Drawing of Tower.

Some reduction might be effected in transverse struts, but an 8″ channel is the smallest allowable in railroad work. As it is, we may employ only $\frac{3}{4}$″ rivets in flanges, a considerable source of trouble and expense, Art. 47 (4).

12 $\frac{7}{8}$″ rivets in transverse diagonals, 24 in longitudinal diagonals, 14 in transverse struts, and 22 in longitudinal struts will carry maximum stresses; we will use not less than these throughout. To put in computed number in *cd′*, for example, would appear weak.

In locating struts and diagonals, we are obliged to deviate somewhat from computed positions. The former must be moved to get clearance top and bottom. The latter, if allowed to intersect on center line of column, will make very large connection plates. Of course the eccentric joint is objectionable. Hardly more so, however, than the lack of compactness in a concentric joint.

Design of Base

Bearing required is either,

$$\frac{66.5+184.0}{.300}=835 \text{ sq.in.},$$

or $$80\% \text{ of } \frac{66.5+184.0+88.6+171.5}{.300}=1362 \text{ sq.in.}$$

Make about 34″×40″.

Prob. 100. What additional weight would be required if compression members were substituted for above diagonals? Also answer same question if construction like Figs. 96*p* and 96*q* be employed.

CHAPTER VIII

SIMPLE RIVETED TRUSS BRIDGES

Art. 101. Truss Bridges in General

In a truss bridge, the principal supports are frameworks or "trusses," instead of girders as in Chap. VI. There are many points of similarity. As in plate girders, spans may be deck, through or half through, the last being comparatively

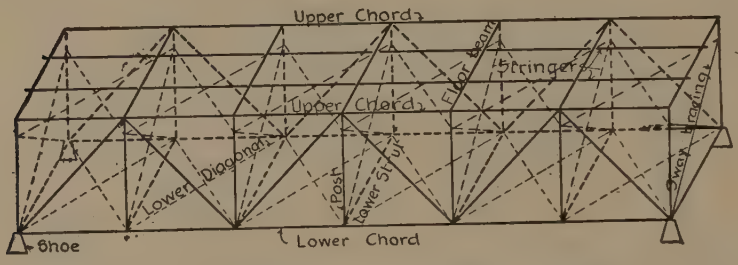


Fig. 101*a*.—Isometric of a Deck Railroad Truss Bridge.

rare. Fig. 101*a* is an isometric line drawing of a deck railroad truss bridge; similarly Fig. 101*b* represents construction for a highway through truss bridge. Sections of typical deck railroad and through highway truss bridges are seen in Figs. 101*c* and 101*d* respectively. We also meet here a new type, the "double decker," with two floors at different levels; often one on a level with each chord.

Much as in plate girders, the ties or planks rest on the stringers, these in turn framing into floor beams carried by the trusses. There should be bracing in the plane of the top chord, "upper laterals"; in the plane of the lower chord,

" lower laterals "; and in the plane of each floor beam, the " sway bracing." The latter in through bridges must be replaced by " knee " and " portal " bracing, Art. 107.

Riveted vs. Pin-connected Trusses

On account of the differences in computation and construction, we shall divide our treatment of the subject into two heads as above. However, they will be indexed together, as traffic to be carried and span length are usually the predetermined quantities.

The advantages of pin-connected trusses are that they have lower secondary stresses, Art. 109, are cheaper, and easier and

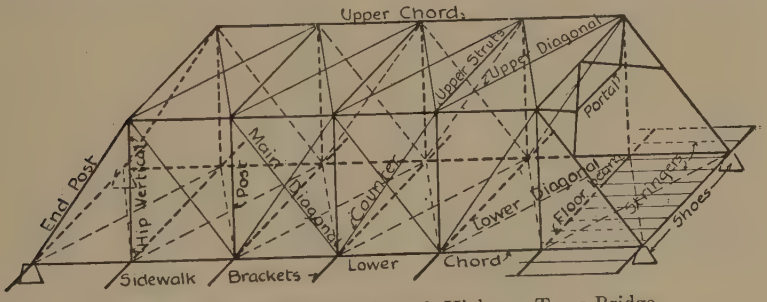


Fig. 101*b*.—Isometric of a Through Highway Truss Bridge.

quicker to erect. The element of time is often of great importance when bridging, by means of falsework, a river liable to sudden floods. On the other hand, the riveted truss is more rigid and less likely to fall if a single member is injured. In Europe, riveted trusses are used almost exclusively. Practice here is to employ riveted trusses for railway bridges of less than 175 feet span. There is a tendency of late to increase this limit. The new Kentucky River High Bridge * of 353 feet span has trusses of the deck Baltimore type with riveted joints. The modern 525-foot span of the Ohio River Connecting bridge near Pittsburgh, Pa., is a riveted structure. In both cases, however, the cantilever method of erection,

* Engineering Record, Vol. 62, p. 774.

putting compression in bottom chord, was partly responsible for this departure from usual practice. Trusses for highway bridges are largely pin-connected. This is because the latter type has in general lighter members; this is quite important where they may have to be hauled from the nearest siding to the site. Another reason is that there is not the same need for rigidity here. And further, as will be shown later on,

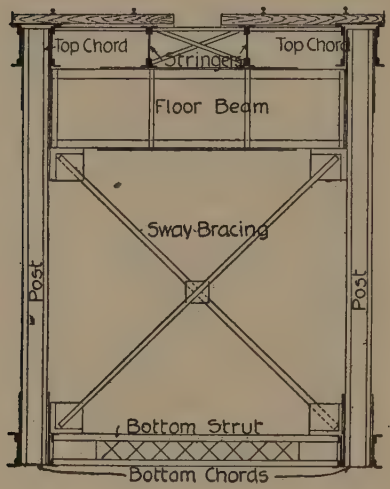


FIG. 101c.

Section of a Deck Railroad Truss Bridge.

FIG. 101d.

Section of a Through Highway Truss Bridge.

somewhat lower standards prevail in highway structures than in railroads.

PONY TRUSSES

The pony truss is a through truss where the depth does not allow room for the overhead bracing. As such structures are short they usually have riveted joints. Their peculiarities of construction are such that we shall allow them a special article, 112.

REFERENCES

From the many good books which treat of the subject of truss bridges in general, we may select:

"Roofs and Bridges, Part III, Bridge Design," by Merriman and Jacoby.

"Theory and Practice of Modern Framed Structures," by Johnson, Bryan, and Turneaure.

"Bridge Design," International Library of Technology.

"Typical Steel Railway Bridges," by Thompson.

"Design of Highway Bridges," by Ketchum.

We will also refer to the following articles in the technical press:

Engineering Contracting, Vol. 30, pp. 225 and 227. Life, weight, and costs.

Engineering Record, Vol. 63, p. 299.

Engineers' Society of Western Pennsylvania, Vol. 20, p. 257; Vol. 22, pp. 45, 141, Highway Bridges.

Journal of the Association of Engineering Societies, Vol. 16, p. 262, Highway Bridges; 30, p. 1, Electric Railway Bridges; 33, p. 180, Economy of Various Types.

In indexing, we shall abbreviate as follows:

Riv. = Riveted.
Pin. = Pin-connected.
Prat. = Pratt.
Balt. = Baltimore.
Warr. = Warren.
Miss. = Miscellaneous.
Park. = Truss with one or both chords curved.
Penn. = Park. with a secondary system.
A. S. = American Society of Civil Engineers.
C. E. = Canadian Engineer.
E. C. = Engineering Contracting.
E. N. = Engineering News.
E. R. = Engineering Record.
R. R. = Railroad Gazette.
S. R. = Street Railway Review.
D. D. = Double Deck Structure.
* = a pony truss.
† = a solid floor for a railroad.
All dimensions are in feet.
Bridges listed in order of span length.

Single Track Steam Railroad

Periodical.	Vol.	Page.	Span.	Truss.	Maximum Depth.	Panel Length.	c/c Trusses	Location.
E. N.	30	517	547	Through Pin Penn	84	30.3	30.0	Louisville, Ky.
E. R.	29	104						
E. R.	23	58, 76	523	Through Pin Penn	..	26.2	25.0	Pittsburgh, Pa.
E. R.	62	153	450	Through Pin Penn	70	28.1	24.0	Miles Glacier, Alaska
E. R.	61	762	420	Through Pin Penn	65	30.0	23.0	Mobridge, S. D.
E. N.	64	196						
E. R.	60	245	413	Through Riv. Penn	55	25.8	20.0	French Riv., Can.
E. R.	50	240	402	Through Pin Penn	60	25.1	22.0	Plattsmouth,
E. R.	53	231	400	Through Pin Penn	65	28.6	22.0	Bismarck, N. D.
E. R.	46	398	388	Through Pin Prat	45	29.8	20.0	Philadelphia, Pa.
E. N.	43	26	361	Through Pin Penn	60	30.1	18.5	Rock Island, Ill.
E. R.	64	524	280	Deck Riv. Prat	50	28.0	22.0	Ione, Wash.
E. R.	41	124	207	Deck Pin Prat	30	Var.	..	Easton, Pa.
E. R.	41	124	189	Through Pin Prat	30	27.0	22.1	Easton, Pa.
E. R.	44	196	170	Through Riv. Prat	30	24.3	17.0	Mexico
E. N.	40	114	150	Through Riv. Prat	28	25.0	17.0	
E. R.	40	524	147	Through Pin Prat	22	21.0	16.2	Atbara, Africa
E. R.	53	231	140	Deck Riv. Prat	18	23.3	12.0	Bismarck, N. D.
E. N.	25	87	136	Through Pin Prat	28	22.6	15.8	Smithville, Mo.
E. N.	41	68	130	Through Pin Prat	28	26.0	16.3	N. P. Standards
E. R.	33	58	128	Deck Pin Prat	23	18.2	..	Big Pipe Creek
E. N.	41	14	120	Through Pin Warr*	11	15.0	16.5	N. P. Standards
E. N.	45	60	120	Through Riv. Warr*	10	12.0	..	N. P. Standards
E. R.	53	231	113	Deck Riv. Warr	11	9.5	7.2	Bismarck, N. D.
R. R.	27	34	100	Through Pin Prat	25	20.0	16.0	B. & O. R. R.
E. R.	49	427	100	Through Riv. Warr*	10	10.0	16.0	

Double Track Steam Railroads

Periodical.	Vol.	Page.	Span.	Truss.	Maximum Depth.	Panel Length.	c/cTrusses	Location.
E. R.	65	424	518	Through Pin Penn	80	32.4	31.5	Havre de Grace, Md.
E. R.	65	633	517	Deck Pin Penn	64	32.3	30.0	Havre de Grace, Md.
E. R.	49	323	498	Through Pin Penn	..	31.1	32.5	Clairton, Pa.
E. R.	44	465	496	Through Pin Penn	82	24.8	33.0	Rankin, Pa.
E. R.	41	516	397	Through Pin Penn	56	28.3	30.3	Port Perry, Pa.
E. R.	62	774	353	Deck Riv. Balt	73 [1]	29.4	31.0	High Bridge, Ky.
E. R.	53	526	255	Deck Pin Prat	31	28.4	19.5 [2]	Havre de Grace, Md.
E. R.	65	102	240	Through Riv. Penn	48	15.0	37.0	Keystone, Pa.
E. R.	45	435	236	Through Pin Penn†	40	14.7	29.3	Buffalo, N. Y.
E. R.	39	371	236	Deck Pin Prat	26	21.4	19.0	Philadelphia, Pa.
E. R.	65	633	235	Deck Pin Prat	35	29.4	19.5	Havre de Grace, Md.
E. R.	62	596	230	Deck Riv. Warr	26	28.8	19.5	Steubenville, Ohio.
E. R.	53	526	195	Deck Pin Prat	31	27.9	19.6	Havre de Grace, Md.
E. R.	41	465	193	Through Pin Prat	33	25.8	30.0	Port Perry, Pa.
R. R.	25	922	173	Through Riv.†	28	13.9	28.4	Stockport, N. Y.
E. R.	64	226	159	Through Riv. Warr	30	19.7	29.5	B. & O. R. R.
E. R.	58	463	159	Through Riv. Balt	30	13.2	16.3	Clyde River, N. Y.
E. R.	52	464	158	Through Riv. Miss	27	9.6	30.4	Shelton, Conn.
E. R.	58	246	140	Through Riv. Warr	35	23.3	..	McKee's Rocks, Pa.
E. N.	44	379	134	Deck Pin Warr	21	16.7	19.5	Richmond, Va.
E. R.	45	290						
E. R.	39	302	118	Deck Pin Prat	11	14.7	18.3	Temperanceville, Pa.

[1] Built around old truss.

[2] On a curve.

Steam Railroad Bridges with More than Two Tracks

Periodical.	Vol.	Page.	Span.	Truss.	Maximum Depth.	Panel Length.	c/c Trusses	Location.
E. R.	46	338	333	D. D. Pin Penn	64	23.8	53.7	Pittsburgh, Pa.
E. R.	49	170						
E. R.	46	364	156	D. D. Pin Prat	26	22.3	53.7	Pittsburgh, Pa.

Electric Railroad Bridges

S. R.	12	329	210	Deck Pin Balt	35	17.5	14.0	Buffalo Electric Ry.
E. N.	62	653	155	Through Riv. Miss†	32	12.9	26.5	Chicago, Ill.
E. R.	49	462	140	Through Riv. Prat	22	20.0	15.0	Greenfield, Mass.
E. N.	38	404	135	Through Pin	24	22.5	23.5	Edgebrook, Mo.
				Deck Prat	23			

Bridges for Combination Traffic

E. N.	67	231				30.0		
E. R.	60	487	668	Through Pin Penn	110	to	35.0	Municipal, St. Louis
E. R.	62	641				48.0		
E. N.	64	85	518	Through Pin Penn	78	28.7	29.7	St. Louis, Mo.
E. R.	61	572						
E. R.	49	582	380	D. D. Pin Balt	50	23.8	19.0	Fraser River, B. C.
E. N.	38	130	254	Through Pin Prat	40	25.4	31.2	Montreal, Can.
E. N.	64	85	245	Deck Pin Park	40	30.7	22.0	St. Louis, Mo.
E. R.	61	572						
E. R.	49	793	239	D. D. Pin Balt	32	17.1	17.5	Woodsville, N. H.
E. R.	49	644	225	D. D. Pin Balt	54	23.2	Var.	Fraser River, B. C.
E. R.	37	384	217	D. D. Pin Balt	..	21.6	..	Rock Island, Ill.
E. R.	49	582	159	D. D. Riv. Prat	29	22.7	19.0	Fraser River, B. C.
E. N.	42	119	89	D. D. Pin Balt†	13	8.9	24.3	Chicago, Ill.

Highway Bridges without Electric Cars

Periodical.	Vol.	Page.	Span.	Truss.	Maximum Depth.	Panel Length.	c/c Trusses	Location.
C. E.	17	509	586	Through Pin Penn	80	32.5	32.5	Elizabethtown, O.
E. N.	48	313	465	Through Pin Park	67	27.3	27.3	New Baltimore, O.
E. N.	45	370	406	Through Pin Penn	50	29.0	26.5	Hamilton, O.
E. N.	37	191	319	Through Pin Penn	45	22.9	21.8	Livermore Falls, Me.
E. R.	59	187	200	Through Riv. Park	28	20.0	31.8	Jersey City, N. J.
E. R.	59	334	157	Through Riv. Balt	27	13.1	20.5	Waterford, N. Y.
E. R.	68	361	150	Through Riv. Park	..	18.7	17.8	N. Y. Standard
E. R.	47	634	143	Through Riv. Prat	18	14.3	15.2	Springville, N. Y.
E. N.	27	122	136	Through Pin Prat	25	16.0	30.0	Chicago, Ill.
E. R.	68	361	125	Through Riv. Warr	20	15.6	..	N. Y. Standard
E. R.	59	351	118	Through Riv. Prat	21	16.9	17.4	Saugerties, N. Y.
E. R.	59	351	..	Through Riv. Park*	13	..	22.5	Ithaca, N. Y.
E. R.	47	423	100	Through Riv. Warr*	12	12.5	32.0	Ohio.
E. R.	65	271	86	Through Riv. Balt*	9	4.3	18.0	Turtle Creek, Pa.
E. N.	61	59	80	Through Riv. Prat*	9	10.0	16.0	Railroad Standard
E. R.	68	361	60	Through Riv. Warr*	7	15.0	17.8	N. Y. Standard
E. R.	32	147	40	Through Riv. Prat*	7	10.0	10.6	Portable for export

Highway Bridges with Electric Cars

Periodical.	Vol.	Page.	Span.	Truss.	Maximum Depth.	Panel Length.	c/c Trusses	Location.
E. N.	68	706	531	Through Pin Penn	93	29.5	36.0	Pittsburgh, Pa.
E. R.	38	224	515	Through Pin Penn	..	25.7	49.0	Braddock, Pa.
E. R.	60	602	515	Through Pin Penn	70	25.8	26.7	Donora, Pa.
E. R.	50	682	454	Through Pin Penn	65	28.4	35.8	Pittsburgh, Pa.
R. R.	25	560	439	Through Pin Penn	79	29.3	44.5	Pittsburgh, Pa.
E. R.	58	686	400	Through Pin Penn	65	20.0	26.0	Wireton, Pa.
A. S.	12	353	360	Through Pin Park	50	27.7	25.7	Pittsburgh, Pa.
E. N.	67	676						
E. N.	62	570	318	Through Pin Park	..	26.5	..	Nashville, Tenn.
E. N.	25	462	317	Through Pin Penn	48	19.8	22.0	Chattanooga, Tenn.
E. R.	38	224	252	Through Pin Penn	41	25.3	49.0	Braddock, Pa.
E. N.	21	474	216	Through Pin Park*	24	13.5	54.1	Boston, Mass.
E. R.	61	273						
E. R.	61	606						
E. N.	25	462	208	Through Pin Penn	38	20.8	22.0	Chattanooga, Tenn.
E. N.	64	33	194	Through Pin Prat	30	32.3	26.0	Waterford, N. Y.
E. R.	61	612	187	Through Riv. Balt	28	..	43.0	Buffalo, N. Y.
E. R.	45	78	168	Through Riv. Penn	32	16.6	25.3	Chippewa Falls, Wis.
E. C.	38	557	90	Through Riv. Warr*	11	9.0	41.7	Niagara Falls, N. Y.

Art. 102. Weights and Economical Dimensions

The length of span and distance between trusses are usually furnished the designer, leaving him free to select such depths and panel lengths as will promote economy. Let M and V represent moment and shear due to the load per lineal inch of bridge, w, acting on a truss of m panels at p equals L, Fig. 102*a*. Calling allowable unit stress S, we have for one panel length, all values in pounds and inches,

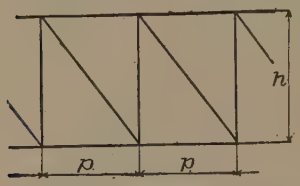


FIG. 102*a*.—Portion of a Truss.

$$\left.\begin{aligned}
&\text{Volume for top chord} = \frac{Mp}{Sh} + \frac{p^2}{40} \text{ *}\\
&\text{“ bottom chord} = \frac{Mp}{Sh}\\
&\text{“ verticals} = \frac{Vh}{S} + \frac{h^2}{40}\text{ *}\\
&\text{“ diagonals} = \frac{V}{S}\frac{p^2+h^2}{h} = \frac{Vh}{S} + \frac{Vp^2}{Sh}
\end{aligned}\right\} \quad . \; . \quad (a)$$

Since p is about equal to h, the sum may be written,

$$\text{Volume for one panel length} = \frac{2Mp}{Sh} + \frac{3Vh}{S} + \frac{p^2}{20}. \quad . \; . \; . \quad (1)$$

The value of h which renders this a minimum may be obtained by placing differential of above expression with regard to h equal to zero, that is,

$$-\frac{2Mp}{Sh^2} + \frac{3V}{S} = 0 \quad \text{or} \quad h = \sqrt{\frac{2Mp}{3V}}. \quad . \; . \; . \quad (2)$$

For a truss of constant depth, the average moment is $wL^2/12$, and mean shear, $5wL/16$, and (2) becomes,

$$h = 0.42\sqrt{pL} = 0.42p\sqrt{m}. \quad . \; . \; . \; . \; . \quad (3)$$

where m is as given below.

* Art. 52.

Substituting in (*a*), value of h from (2) and adding, we have for the volume of one truss for one panel length,

$$\frac{4.1}{S}\sqrt{MVp}+\frac{1.25}{S}\sqrt{\frac{V^3p^3}{M}}+\frac{Mp}{60V}+\frac{p^2}{40}. \quad . \quad . \quad . \quad (4)$$

For a distributed load, w, on entire bridge, the average moment on each truss is $wL^2/24$, and mean shear is $5wL/32$. Substituting and multiplying by $2L/p$ to obtain volume of two trusses, everything in inches,

$$\text{Vol.}=\frac{0.67}{S\sqrt{p}}wL^{\frac{5}{2}}+\frac{0.76}{S}w\sqrt{pL^3}+.05pL+.009L^2.$$

Next let L = span in feet;

d = center to center of trusses in feet. For a highway bridge, add one-half the total width of sidewalks;

p = panel length in feet;

m = number of panels $= L/p$;

n = number of stringers between which the load is divided;

w = load above stringers on both trusses per lineal foot. If concentrated loads be employed, make an estimate of an equivalent uniform load;

S = average allowable unit stress for short members in pounds per square inch;

T = total weight in pounds of steelwork in bridge including shoes, trusses, laterals, floor beams, and stringers.

Using above units, adding 30% for details, and 20% for dead loads, and taking the weight of the steel at 0.28 pounds per cubic inch, above formula becomes,

$$\text{Weight of trusses}=\frac{3.4wL^{\frac{5}{2}}}{S\sqrt{p}}+3.8\frac{w}{S}\sqrt{pL^3}+3pL+0.55L^2. \quad (5)$$

The weight of the built floor beams and stringers by the results of Art. 71, are respectively for railroad bridges,

$$11Ld\sqrt{\frac{wd}{Sp}} \quad \text{and} \quad 11pL\sqrt{\frac{nw}{S}}.$$

To allow for extras, dead loads, laterals, and so forth, above coefficients will be increased to 20 and 25 respectively. Then,

$$T=3.4\frac{wL^{\frac{5}{2}}}{S\sqrt{p}}+3.8\frac{w}{S}\sqrt{pL^3}+3pL+0.55L^2+20Ld\sqrt{\frac{wd}{Sp}}+25pL\sqrt{\frac{nw}{S}}. \quad (6)$$

$$\frac{\delta T}{\delta p}=-1.7\frac{wL^{\frac{5}{2}}}{S\sqrt{p^3}}+\frac{1.9w}{S}\sqrt{\frac{L^3}{p}}+3L-10Ld\sqrt{\frac{wd}{Sp^3}}+25L\sqrt{\frac{nw}{S}}=0.$$

Whence the value of p which renders T a minimum is that derived by a solution of the equation,

$$\left(3\sqrt{\frac{S}{w}}+25\sqrt{n}\right)p^{\frac{3}{2}}+1.9p\sqrt{\frac{w}{S}L}=1.7\sqrt{\frac{w}{S}L^3}+10\sqrt{d^3}. \quad (7)$$

For most cases we may ignore the first, third, and last terms and write,

$$p=\frac{L}{6}\sqrt[3]{\frac{w}{nS}}. \quad (8)$$

For railroad bridges, let us estimate the average value of w/nS to be $\frac{1}{4}$. Substituting in (6), we obtain,

$$T=L^2\left(12\frac{w}{S}+0.85\right)+L\left(20\sqrt{\frac{d^3}{p}}+25p\sqrt{n}\right)\sqrt{\frac{w}{S}}. \quad (9)$$

Rolled beams are used for highway stringers and coefficient 15 should be substituted for 25 in (6) and (7); (8) then becomes approximately

$$p=\frac{L}{4}\sqrt[3]{\frac{w}{nS}}. \quad (8a)$$

On account of the usual large values of (d) this may not be employed; we may use it however to substitute in (6a) giving for the weight when $\frac{w}{nS}=\frac{1}{60}$

$$T=L^2\left(14\frac{w}{S}+0.70\right)+L\left(20\sqrt{\frac{d^3}{p}}+15p\sqrt{n}\right)\sqrt{\frac{w}{S}}. \quad (9a)$$

While this proof does not in general apply to trusses with a secondary system, braced at intermediate points, (9) and (9*a*) may be used for approximations.

Practical Modifications of Theoretic Depth

The economical panel length having been first determined by (7) or (8), we must use a factor of the total length, L, of about the theoretic value.* We next compute economical depth from (3). Values somewhat above this will be best for bridges with bent chords. For a through truss with overhead bracing, there is a minimum practical depth. This will be, (*a*), amount base of rail or surface of road is above bottom chord, plus (*b*), clearance height, plus (*c*), vertical amount allowed for overhead bracing. While (*a*) may be made small or even negative by suspending floor beams below joint, there are objections to this method and it is better to allow $p/8$. (*b*) will vary, a mean value being 22′0″ for railroads and 14′0″ for highways. (*c*) may be taken as not less than one-fortieth of span. Approximate dead weight may now be computed by the aid of (9) or (10).

As these formulæ are approximate only and depend on many practical conditions, they should be tested. (2), (3), (7), and (8) may be verified by trying slight departures from theoretical amounts; if cost is lessened, try farther in the same direction. After design is made, weight should be checked, and, in important work, more particularly in long-span bridges, weight should be calculated from detail drawings.

Width of Bridge

Transversely, deck trusses should be not less than one foot outside of outside rails to provide some stability in case of derailment. Through trusses must not interfere with clearance lines, the usual spacing being 16 feet for single- and 30 feet for double-track railroad bridges. Widths of highway

*We have not taken into account shopwork or minimum sizes, which tend to increase panel length. These relations should not be accepted without trial and in general it will be better to assume in starting values taken from a similar structure.

bridges are governed by traffic conditions. Sometimes it may be necessary to increase spacing of trusses to provide lateral stability. Should these distances be less than one-twelfth of the span, the strength of the bridge under lateral loads and the stiffness of the top chord should receive careful attention. The fraction, *width divided by span*, should never be less than one-twentieth.

REFERENCES GIVING THE WEIGHT OF TRUSS BRIDGES

Engineering News, Vol. 44, pp. 132, 427; Vol. 45, pp. 60, 433; Vol. 49, p. 482.
Engineering Record, Vol. 57, p. 450.
Engineering Contracting, Vol. 30, p. 225.

Prob. 102. A single-track railroad bridge with Pratt trusses is 200 feet long and 18.0 feet center to center of trusses. It is designed for a load of 7000 pounds per lineal foot with allowable stresses of 10,000 pounds per square inch. Compute approximate weight and compare with those obtained from references above.

Art. 103. Forms of Trusses

The value of the fraction, *economical depth divided by the span*, will vary with a number of quantities as indicated by the equations. Roughly, it lies between one-fifth for short spans and one-eighth for long. Panel lengths are, for bridges over 200 feet long, from 20 to 30 feet with a recent tendency towards even higher values; as indicated by formula (8) of preceding article, they are largely functions of the length of span. The theoretic angle for the diagonals considered alone is 45°; this may be proven as for bracing, see Art. 76; while slope may be made somewhat steeper, it is desirable not to go too far. For the longer spans then, we find either Baltimore or Pennsylvania trusses common, as they keep the economical inclination of the diagonals without necessitating long panel lengths.

For short spans, the simpler trusses will do. There is a considerable saving of material in increasing the depth of the truss towards center, hence the use of the parabolic, Parker, and Pennsylvania trusses. Not only are the web stresses low, but the chord section required may be made nearly constant. On the other hand, shop costs are increased by the bending, skew cuts and lack of similarity of parts. We find then that for small spans, the parallel chords are, on the whole, cheaper.

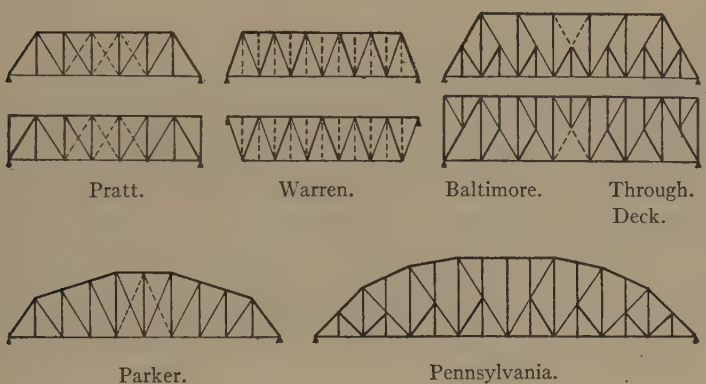


FIG. 103*a*.—Best Forms of Common Trusses.

There is a slight tendency to favor the Warren truss for riveted spans and the Pratt for short pin-connected bridges. We may then construct the following table, which represents best general practice:

Span.	Best.	Second Choice.
100 to 150 feet	Warren Riveted	Pratt Riveted
150 to 200 feet	Warren Riveted	Pratt Pin-conn.
200 to 300 feet	Parker Pin-conn.	Baltimore Pin-conn.
300 to 400 feet	Pennsylvania Pin-conn.	Parker Pin-conn.
400 to 500 feet	Pennsylvania Pin-conn.	

Pratt Type

The economical arrangement of the members and the simple joints give this type a deserved preference. Sometimes counters are omitted and the corresponding main members designed to take stresses of either kind, that is, *counterbraced*. This seems to be steadily gaining in favor, as it makes a more rigid

Fig. 103*b*.—Riveted Deck Pratt Truss for Norfolk and Western R. R. at Shepardstown, W. Va.

(1) Through. (2) Deck. (3) Deck.

Fig. 103*c*.—Alternative Methods at End.

structure. On the other hand, it has the undesirable effect of subjecting the member to alternating stresses.

Other arrangements in end panel are shown in Fig. 103*c*; comparative estimates are the best method of deciding on advisability of these modifications with chances in favor of the trusses seen in Fig. 103*a*.

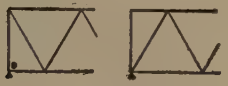
Warren Type

The vertical members shown dotted in Fig. 103*a* are employed to reduce panel length, slenderness ratio, and stresses due to own weight. The direction of the diagonals may be reversed, the end panel with these changes appearing as in Fig. 103*e*.

Baltimore Type

The arrangement of the diagonals may be like the Pratt or Warren trusses or any of their modifications. Also the sub-diagonals may be compression members as in Fig. 103*a*

Fig. 103*d*.—Riveted Through Warren Truss Bridge for Virginia Railway, at Norfolk, Va. McClintic-Marshall Construction Co., Rankin, Pa.



Deck.
Fig. 103*e*.

Through.
Fig. 103*f*.

Modifications for Warren Trusses.

or tension members as in panel seen in Fig. 103*h*. The Pratt arrangement may have counters as shown dotted in Fig. 103*h* or the diagonal may be counterbraced where necessary.

Parker Type

The curve of maximum economy probably approximates a parabola with vertex at the center of the span. Trusses in which one or both chords are parabolas are occasionally seen. However, there are three disadvantages to such construction:

(*A*) Counters or counterbracing will be required in each panel.

(*B*) Shop work will be high on account of skew cuts, bent plates and varying members.

(*C*) Lack of depth at first intermediate panel points prevent effective bracing. This last objection is extremely important, and hence we find the parabolic form confined in the best practice to deck and pony trusses. They have already been indexed as Parker trusses.

To overcome (*C*) in the typical Parker truss we make height at hip joint, Fig. 103*a*, sufficient for proper stiffness, Art. 102.

FIG. 103*g*.—Riveted Deck Baltimore Truss at Kansas City, Mo.

A parabola may then be run between the hip joints on either side, having as its vertex at the middle the economical height. Members should be straight between panel points; to make them curved is not only expensive, but introduces large secondary stresses, Art. 109. The above method partially eliminates disadvantage (*A*). To lessen shop work, where architectural treatment is not important, the parabola may be replaced by broken lines approximating thereto as seen in Fig. 103*a*.

FIG. 103*h*.
Baltimore Truss with Counters.

The modifications of the Pratt truss may be employed here. The deck type is comparatively infrequent, as it is seldom that, in the long spans for which it is most suitable, sufficient clearance can be obtained underneath.

Pennsylvania Type

This is simply an application of the principle of the secondary system to the Parker truss. On account of the small depth at the end, the secondary system may be omitted for the first few panel lengths. The latter, it will be noted, should be kept constant.*

Extra pieces are often inserted to reduce stresses due to own weight and lessen unsupported length. See Fig. 103*j*. The *collision strut*, Fig. 103*k*, not only supports end post, but strengthens member against a blow from a derailed train, hence the name.

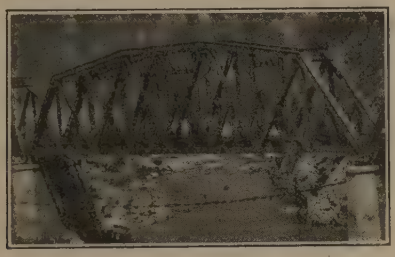


Fig. 103*i*.—Riveted Through Pennsylvania Truss over Feather River, Plumers Co., California, for Western Pacific R. R. McClintic-Marshall Construction Co., Rankin, Pa.

Fig. 103*j*.
Modification of Secondary Systems.

Fig. 103*k*.
Truss with Collision Strut.

Prob. 103. A deck Pratt truss is 32 feet deep, has 10 panels at 24 feet each, and is subjected to live panel loads of 80 kips each, with dead of 24 and 12 at top and bottom respectively. Find the sum of the products obtained by multiplying stress in each member by the corresponding length, for the members which vary in the three modifications shown above. Multiply compression by the factor 1.6 to allow for increased metal required.

* The Municipal Bridge at St. Louis, Mo., is a notable exception. See references. Here the panel lengths were increased towards center to keep inclination of diagonals about the same. On account of the high cost of the trusses for this long span, their economy was far more important than that of the floor system.

Art. 104. Sections and Joints for Riveted Trusses

We may divide this article into two parts: first, trusses where the joint is made by one plate; second, where made by two. From the nature of the sections which must be employed, the former is limited largely to the smaller structures and those carrying light loads.

The requirements for suitable members of trusses are as follows:

(*A*) *Economy of Material.* In tension, shapes should be such that the deduction for rivet holes is kept as low as possible. For compression, so locate the metal that the radius of gyration of a given area is made a maximum.

(*B*) *Low Shopwork.* Keep riveting and number of pieces to a minimum consistent with full preservation of strength. Avoid also bent work and skew cuts.

(*C*) *Easy Splices.* Members in a straight line should be designed to fasten together readily. This is sometimes difficult to accomplish when the stress changes considerably.

(*D*) *Economical and Efficient Joints.* The joint is as important in strength and economy as the member. Consideration of the manner in which its connections are made will be found to be very important.

(*E*) *Accessibility.* Although closed sections are sometimes seen in old structures, they are no longer allowed in modern bridge design.

(*F*) *Drainage.* Sections must not hold water. Pockets are sometimes filled with concrete or may have holes punched at the bottom. It is better, however, if they may be avoided altogether.

Cross Sections of Members for Riveted Trusses with Single Plate Joints

Below are shown those combinations which are convenient and fairly central for the small stresses involved. Members of one or two plates are no longer in favor; if thin, they bend out of line; if thick, they require too many rivets. For top chords and end posts, all but (1) may be employed, (3) to strengthen (2) as (5) does (4). The last two are objectionable

on account of the lack of stiffness in projecting plate. If used, it should not extend from the angles more than twenty times the thickness of plate. On the other hand, the latter is utilized for connections and it should be sufficient for this purpose. In (2) and (3), the distance between angles is the thickness of joint plates. Between the latter at intervals of 18 to 24″ for compression members, and 24 to 42″ for ties, are located " washer fillers," Fig. 37*k*. These act as spacers and have a radius not greater than edge distance.

For the bottom chord (3) is objectionable, since it violates principle (*F*), if plate be underneath. (2), (4), or (5) may be used, since the main disadvantage of the last two are now removed. In the web members, (1) may serve for a small

(1) (2) (3) (4) (5)

Fig. 104*a*.

tension. In compression, it is objectionable on account of its lack of stiffness and the eccentricity of its connection to the plate. For the compressions and the larger tensions, use (2).

Cross Sections of Members for Riveted Trusses with Double Plate Joints

Here we first determine, often after several preliminary designs, proper spacing of plates, and then all sections are arranged to go either outside or inside. Usually posts are made as small as convenient and economical, and then plates are placed just outside.

The top chord, including the end post, also goes outside. Here two channels, either rolled or built, with a top cover plate are used almost exclusively. To provide for increase of stress, the weight of the channels is altered, or additional side plates placed outside of the section, Fig. 104*c*. In making changes, it is desirable to move the center of gravity as little as possible. The cover plate should not be too thick, since

it does not bear its full share of the load, Art. 50 (*d* 4); the clause in Art. 50 (*e* 4), will be found to determine its minimum thickness.

For the remaining members, we may employ either the two-channel or the four-angle section in the I form. The webs of the channel and legs of outstanding angles are placed parallel to plane of truss. The former may be rolled or built or have flanges turned out or in. The latter may be united by a solid plate, by batten plates, or by lacing, the last two being also available for channels. Batten plates should not be used where there is a possibility of much compression in a member. When placed horizontally, the four angles and plate sections have the grave fault of retaining water. The built I has not as economical a disposition of metal as the two channel, but its percentage of details is smaller.

FIG. 104*b*.

FIG. 104*c*.

Top Chord Sections.

To increase area of two channels, plates may be placed on the flange side, as it is obviously necessary to keep distance between the backs the same. The built I should have distance back to back of angles the same throughout; for the smaller stresses, we may use light angles and lacing; for larger amounts, a plate can be inserted; and then the thicknesses of plates and angles may be increased.

SPLICES FOR RIVETED TRUSSES

The subject of splices in general has been considered in Art. 59. For case in hand, they may be necessary for top and bottom chords. Former should be milled and have two or three rows of rivets on each side of joint in each splice plate. Tension joints, on the other hand, must be figured, care being taken that the strength of net section is *fully* maintained. Rivets in the first row from splice should also be arranged to take care of net section of splice unless the latter has 15%

excess. In either, splice should occur at a panel point of the bracing or as near it as is practicable. See Figs. 104*d* and 104*e*.

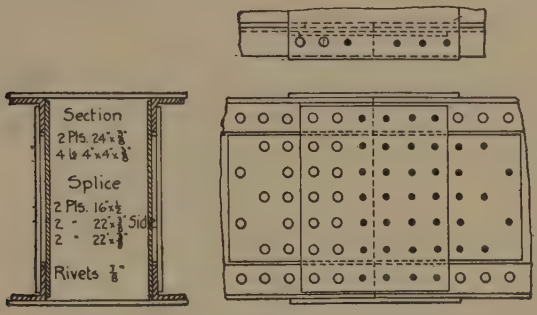


FIG. 104*d*.
Two Built Channels.

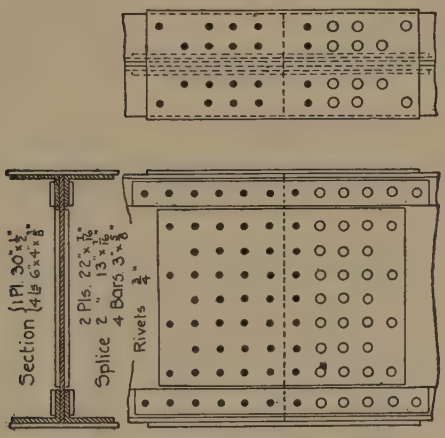


FIG. 104*e*.
A Built I.

Splices.

JOINTS FOR RIVETED TRUSSES

These have also been considered in Art. 59. We have here additional complications, the connections to the floor system and bracing. We will consider these later, but designer must carefully bear them in mind in laying out the work. Rolled

channels and web plates of built channels are connected by rivets through the web, Fig. 104*g*. Angles are connected direct for one leg with a lug for the other if practicable, Fig. 104*f*. In latter, connection would probably be improved if plate were enlarged to dotted line.

Plates may be shipped loose or riveted to the chords, preferably the latter. The outside dimensions of the members which

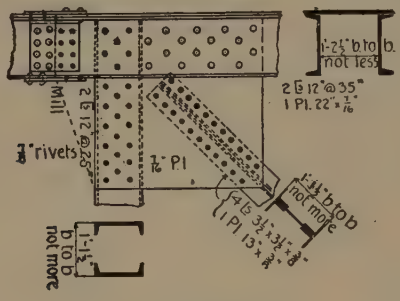


Fig. 104*f*.
At Top Chord.

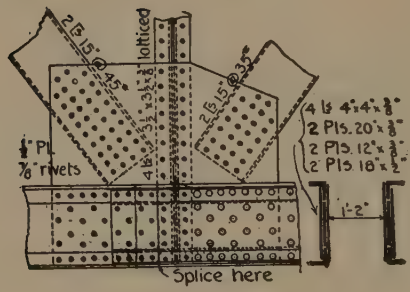


Fig. 104*g*.
On Lower Chord.

Typical Joints.

go inside of the connection plates should be $\frac{1}{8}''$ less than distance in the clear between the plates.

Prob. 104. Splice, to preserve strength of net section, 2 built channels each of 1 plate, $24''\times\frac{1}{2}''$, and 2 Ls, $6''\times6''\times\frac{1}{2}''$. Use medium steel and allowable stresses of Art. 92. Rivets are of $\frac{7}{8}''$ diameter.

Art. 105. Floors for Riveted Truss Bridges

Open Floors for Railroads

These are much like those for half through girder bridges, Arts. 77 and 78. Panel lengths of 20 to 30 feet represent best practice, hence we find more built I-beam stringers. If width of top flange in stringer be less than $\frac{1}{25}$ the span, it must be stiffened. A better bridge will result if bracing be inserted for somewhat wider flanges, and this is generally done. Also like half through girders, it is customary to employ a small floor-beam gusset connecting to the post for through structures. However, except in pony trusses, it is not an important part, serving merely to brace the post and it may be omitted without injury. Connection is usually made through joint plate to posts, with a few rivets in bottom chord. Opposite this connection, a diafram, Fig. 61*m*, should be inserted in a two-channel post; or a solid plate put in a latticed angle section. The purpose of this precaution is to ensure the proper distribution of the load. Bottom of floor beam is preferably placed about on a level with the bottom chord in order to make a more effective lateral system. In deck bridges, the ties sometimes rest directly on the top chord, Fig. 105*b*. This is somewhat objectionable on account of the combined stresses. Another design is to attach floor beams to posts just below top chord and then use a floor system like that previously outlined for the through structure, Fig. 105*c*. However, such a design would mean that wind laterals must be located far below the top chords with which they act. To avoid this, we may place stringers on top of floor beams, Fig. 105*d*. The objection to this is that it necessitates X bracing at end of stringers and deprives the floor beam of intermediate lateral support. Still another arrangement is to put the floor beam on top of the top chord and frame the stringers into it, Fig. 105*e*. This requires bracing for the end of the floor beams. Similar forms for double-track bridges will be obvious except the corresponding form for 105*b* which is shown in Fig. 101*c*.

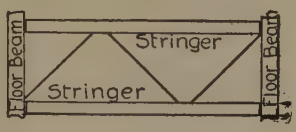


Fig. 105*a*.—Bracing for Stringer.

Solid Floors for Riveted Trusses for Railroads

It will be quite seldom that conditions will be such as to demand a solid floor for a truss bridge. On that account, there is no well recognized standard practice. The floor usually takes the form of troughs extending from truss to truss, and is suspended from, rests upon, or is framed into the loaded chord,

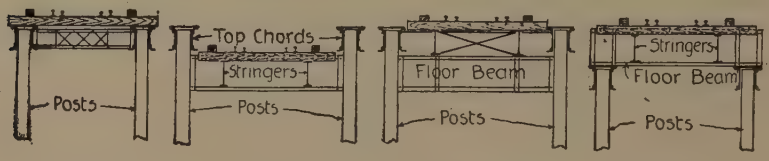


Fig. 105*b*. Fig. 105*c*. Fig. 105*d*. Fig. 105*e*.

Arrangement of Floor for Deck Bridges.

preferably the latter. Either the first or last are carried by one angle fastened to each vertical web. In second case, stiffeners should be used at ends of troughs.

Floors for Riveted Trusses for Highways

Flooring, stringers, and floor beams are like those for highway plate girder bridges as already considered in Art. 84. About the only difference worth mentioning is in its connection to the truss. As in railroad truss spans, the base of the floor

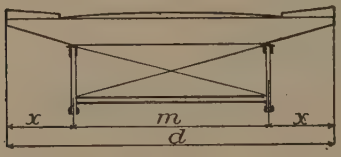


Fig. 105*f*.

Highway Floor Beam.

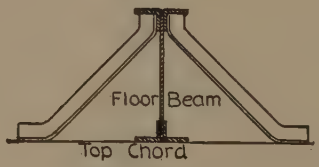


Fig. 105*g*.

Bracing of Floor Beam.

beams is best located about on a level with the bottom chords in a through bridge. This point is not as important as in railroad bridges, since the lateral system and the main trusses are not both heavily stressed at the same time.

The connection between post and floor beam in through

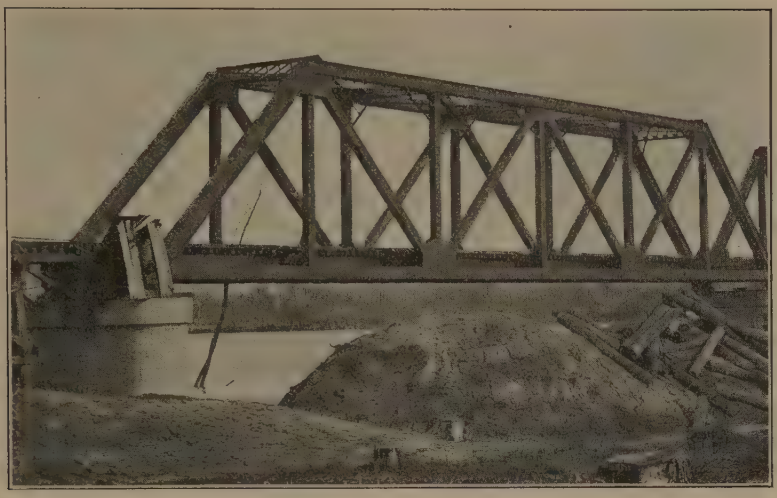


FIG. 105*h*.—Through Riveted Warren Truss Bridge over Wabash River at Mt. Carmel, Ill. N. Y. Central Lines. McClintic-Marshall Construction Co., Rankin, Pa.

FIG. 105*i*.—Through Riveted Pratt Trusses for Electric Cars over Baltimore & Ohio R. R., at Rankin, Pa.

bridges is a source of considerable trouble. It is usually made by the typical two angles. If the two-channel section is employed, a diafram opposite it becomes necessary. These details determine size of post which cannot well be less than 9″ and ought to be more. Whether the two-channel or built I section is employed, the connection of the sidewalk bracket depends largely on rivets in tension, and is a continual worry to the designer. One alternative is to box around post, thus connecting top flanges of floor beam and bracket.

Deck trusses might be similarly arranged with a connection immediately below the top chord. Partly on account of the troubles just outlined and partly to get an effective lateral bracing, it is better to arrange as seen in Fig. 105*f*, the floor beam being braced to the top chord as seen in Fig. 105*g*. It will be noted that trusses need not be placed under curb, but can be moved to secure maximum economy. Let w be the load per linear unit, and x, d, and m as seen in Fig. 105*g*. Assuming load to be dead and beam of uniform section,

$$\tfrac{1}{2}wx^2 = \frac{wm^2}{8} - \tfrac{1}{2}wx^2, \quad \text{or} \quad x = 0.35m = 0.20d.$$

In practice, beam is often of non-uniform section and load is live and heavier in the center; but these counterbalancing features will not alter the relation much and it may be used in the first trial for the economical position.

Prob. 105. Under above conditions, what value of x produces the least total area of the moment curve?

Art. 106. Shoes and Their Connections

For spans suitable for trusses, say those exceeding 100 feet, shoes with pins should be employed; Type 4 for fixed and Type 5 for expansion end, Art. 61. The proper diameter of the pin is about 1/1200 of the span plus 3″. This is an empirical rule which may be varied to secure the best design. Fig. 106*a* shows the arrangement for a vertical end post; Fig. 106*c* the inclined; while 106*b* gives a section of either. Another method, applicable in both cases, is seen in Fig. 106*d*.

The length of bearing required, t, for the maximum load, P, for a given unit bearing stress, S_{bg}, and diameter of pin, d, is determined as in rivets,

$$t = P/S_{bg}d.$$

To reduce moment on pin, it is desirable to keep these bearings as nearly in line as possible. For the same reason, bearing of inclined end post should be between that of shoe and bot-

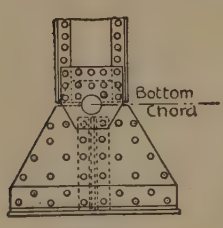


FIG. 106*a*.

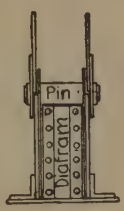


FIG. 106*b*.

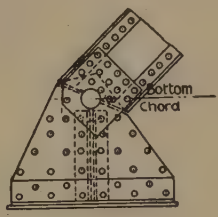


FIG. 106*c*.

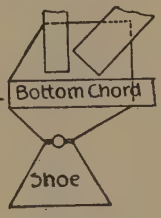


FIG. 106*d*.

Shoes for Truss Bridges.

tom chord. Often we find the hinge plates omitted and the pin plates right in line but kept about $\frac{1}{4}''$ apart while a half hole is placed in each member, Fig. 106*e*. This is open to the objection that there is nothing but the weight of the bridge to resist uplift. The hinge plates should clear transversely by at least $\frac{1}{8}$ inch on a side, to be tripled if there are countersunk rivets. This arrangement produces small moments; however, pin should be tested by the methods of Art. 120.

Trusses, like the half through plate girders, Figs. 77*e* and 77*f*, may have reaction stringers or end floor beams. They are somewhat similar to those considered in Chap. VI. We may now make the schedule of connections for joint at shoe given below, adding on the bracing to be treated in our next article. Note that a floor beam connection on one side on a highway may involve a bracket on the other; also that for some deck trusses these connections will be above top chord and so not affect the shoe. This

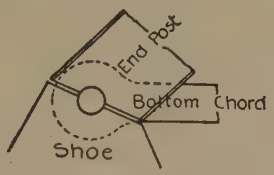


FIG. 106*e*.
Joint without Hinge Plates.

is not altogether an advantage as the floor beam has a decided stiffening action.

CONNECTIONS FOR SHOE

	Type.	Location of Shoe.		Connections.		
(*A*)	Deck	Upper chord	End floor beam	Floor beam	Top diagonal	Sway diagonal
(*B*)	Deck	Lower chord	End floor beam	End Strut	Bottom diagonal	Sway diagonal
(*C*)	Through	Lower chord	Reaction stringer	End Strut	Bottom diagonal	
(*D*)	Through	Lower chord	End floor beam	Floor beam	Bottom diagonal	

Where the end floor beam is employed, it also acts as a strut for the laterals, which usually attach to the top or bottom flange of the former. When it is dropped below level of chords,

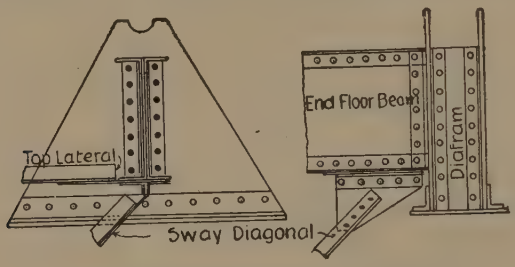


FIG. 106*f*.—Detail Around Shoe. Case (*A*).

shoe must have a depth sufficient for proper connection. Fig. 106*f* illustrates one detail around shoe for truss seen in Fig. 103*c*, (*3*), provided floor beam is so lowered. Plates are shop-riveted to floor beam and field riveted elsewhere. Care must be taken that sway diagonal does not interfere with masonry. As designed, a pedestal will be necessary. Floor beam is often dropped lower, but this renders the driving of the field rivets for top lateral difficult. If diagonals are in plane of top of floor beams, we may have details seen in Fig. 106*g*.

Case (*B*) may be handled as shown in Fig. 106*h* or the strut may connect in the same way that the intermediate struts do to the top chord, Art. 107.

Case (*C*) may also be represented by Fig. 106*h* if we omit sway bracing. Bottom laterals might be fastened underneath strut instead of on top.

There are two distinct methods of solving Case (*D*). When the floor beam is below the bottom chord, we may use detail

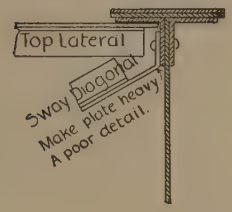


FIG. 106*g*.—Connection of Floor Beam and Sway Diagonal.

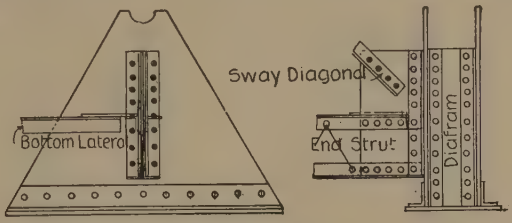


FIG. 106*h*.
Detail at Shoe. Case (*B*).

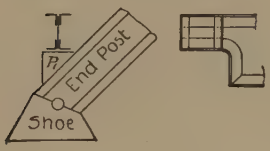


FIG. 106*i*.

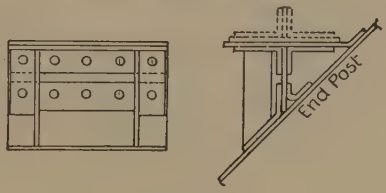


FIG. 106*j*.
Detail of P_1.

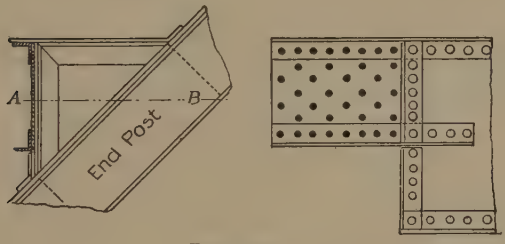


FIG. 106*k*.

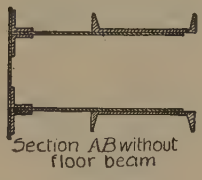


FIG. 106*l*.

Details of Notched End Floor Beams.

of Fig. 106*f*, except that sway bracing is omitted. When floor beam is above pin and the end post is inclined, a very frequent occurrence, in both riveted and pin trusses, the joint may be treated as in Figs. 106*i*, 106*k*, or 106*l*. These are very expensive in the shop. The notched floor beam is particularly objectionable; nevertheless it is quite common in pin trusses where

its shortcomings will be dealt with at length. For riveted trusses only, we may stop the end post and connect floor beam to bottom chord, dropping the shoe down below, see Fig. 106*d*. For pin trusses, a better design would be to drop the end floor beams and connect to shoe. In this case, stringers would rest upon the floor beam, rather than frame into it.

Art. 107. Bracing

The matter on this subject in Arts. 81 and 84 may refer with few changes to riveted bridges. Single angles are the common section for diagonals, but 2 or 4 are also used, the latter being quite common for the unloaded chord. The angles are then formed into a latticed I section, back to back of the angles equaling the depth of the chord. This same section is the accepted design for struts, when the floor beam does not act in this capacity. Web is usually placed normal to chords. A solid floor is equivalent to bracing in its own plane.

The essential points in the design of truss bracing are as follows:

(*a*) Complete bracing must be provided for points at intervals not exceeding unsupported length assumed in computations.

(*b*) Avoid secondary stresses, Art. 109, by arranging members to intersect at center lines, provided connections are such as to transfer the stress directly to the center line of member.

(*c*) Strive to secure compact connections; when this conflicts with (*b*), use judgment as to best course.

(*d*) Avoid secondary stresses also, by keeping struts and diagonals as nearly as possible in the planes of the members of the truss with which they act, namely: upper chord for upper laterals, lower chord for lower laterals, and posts for sway bracing.

(*e*) Attachment to the stringers serves to carry tractive stresses, decreases those due to own weight, and stiffens laterals and stringers.

(*f*) Until sufficient practical experience has been attained, it will be wise to compute stresses due to own weight in light members; it will prevent the use of over weak sections.

Bottom Laterals of Deck Bridges

The struts in the bottom lateral systems of deck bridges should be kept in the plane of the chords. They may be connected to the trusses by the usual pair of angles, Fig. 107*c*. It is much better to employ plates field riveted to strut top and bottom, Fig. 107*d*, and riveted to outstanding flanges of chords and diagonals. Fig. 107*c* would be better if lateral

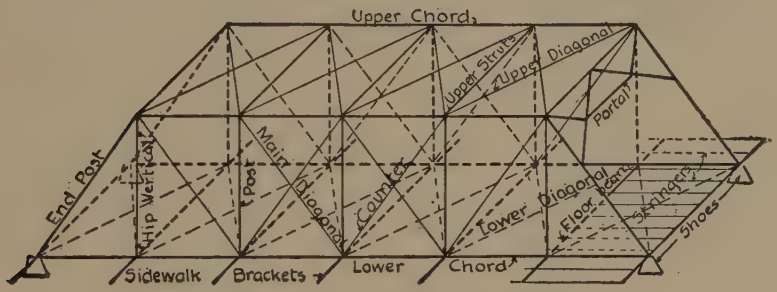


Fig. 107*a*.—Bracing for Through Bridge.

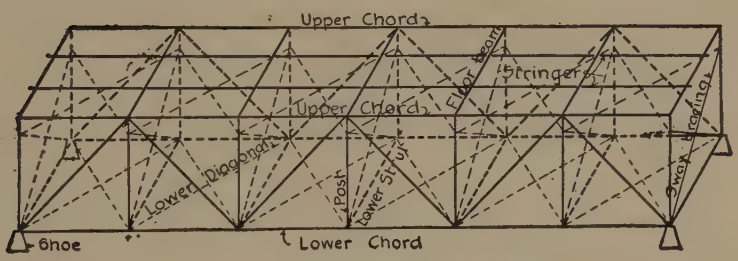


Fig. 107*b*.—Bracing for Deck Bridge.

plate were connected to chord by horizontal angles. If a single angle is used for the diagonals, it is preferable to place it on lower plate with vertical leg up. However, its location may be influenced by an easy and efficient joint at shoe. In this connection, note that while it is desirable to keep laterals in the same plane, they may be bent if additional shop work is not an objection.

Top Laterals of Deck Bridges

(*A*) *Floor beams omitted, ties resting directly on top chord.* Nothing less than the latticed I section should be acceptable here for struts. Their depth should be about that of the chord. It may be made a little less to decrease interference with the ties, as shown in Fig. 107*e*. For this same reason, laterals, if of single angles, are better placed in the plane of the bottom of the strut.

(*B*) *Floor beams resting on the top chord.* Diagonals, if latticed I's, should be of depth and in plane of top chord; if

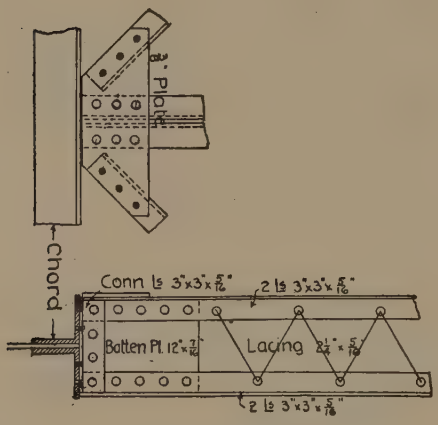


Fig. 107*c*.
Detail of a Strut.

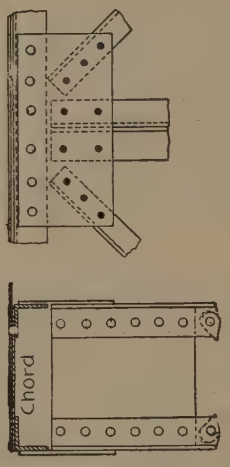


Fig. 107*d*.—Alternative Design of End of Strut.

of a single angle, in the plane of the top of the upper chord; in either case, the floor beams serve as struts.

(*C*) *Floor beams framing into posts, stringers into floor beams.* Diagonals here are of single angles turned up in the plane of the bottom of the floor beams.

(*D*) *Floor beams framing into posts, stringers on top of floor beams.* Diagonals here are of single angles in plane of the top of the floor beams. Angles should have vertical leg turned down.

In (*C*) and (*D*), angles should connect to stringers as in Art. 81.

Sway Bracing in Deck Bridges. See Fig. 101*c*

The diagonals which form the transverse bracing in the plane of each pair of posts are usually composed of one or two angles, field riveted at intersection and to plates at end; the latter, either field or shop riveted, connect posts with struts or floor beams.

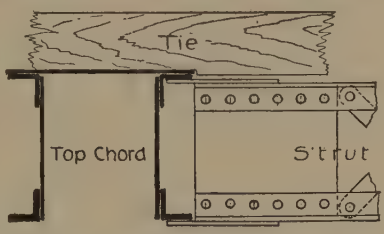


Fig. 107*e*.—Connection of Lateral Strut to Top Chord.

It is customary to assume that entire stress due to wind is carried by lower laterals, upper laterals, and end sways. This leaves the intermediate sway bracing without stress. It is usually made with one-quarter to one-half of the strength of the end frames.

Bottom Laterals, Through Bridges

These are placed in the plane of the bottom of floor beam. Where latter is dropped below the joint, this means a distinct violation of (*d*) above. There is no excuse for such construction in either pin-connected or riveted trusses.

Top Laterals, Through Bridges

These are made much like the bottom laterals for deck bridges. However, there is even more need for strength and stiffness since they serve in a way as the lacing for the top chord.

Sway Bracing, Through Bridges

To allow the passage of traffic, bracing must be partial, Art. 99. For short spans where depth is just sufficient for passage, we find intermediate frames of the form shown in Fig. 107*g*. For deeper trusses, the additional room may be utilized to insert more rigid types, examples of which are seen in Figs. 107*f* and 107*h*. As indicated under deck bridges, these members

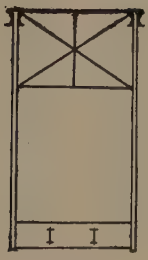


FIG. 107*f*.

FIG. 107*g*.

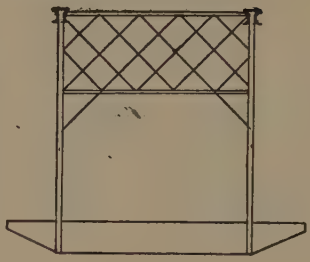


FIG. 107*h*.

Intermediate Bracing for Through Bridges.

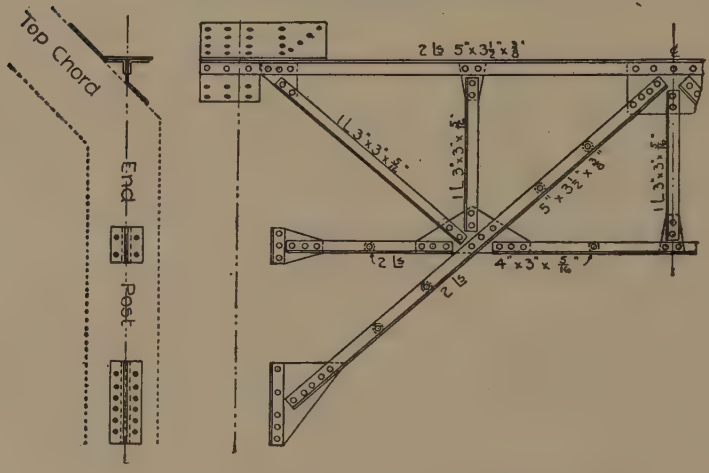


FIG. 107*i*.—Typical Single Plate Portal for a Highway Bridge.

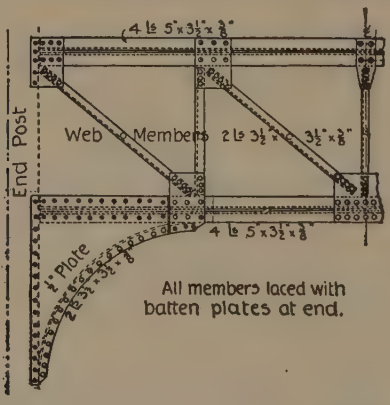


FIG. 107*j*.—Typical Double Plate Portal for a Railroad Bridge.

are not assumed to carry any stress.* At end, however, conditions are quite different, and we shall consider carefully the sway bracing at this point called the

PORTAL

Line drawings of the principal types of portals are given in Art. 99 under the head of partial bracing. In choosing a type, consider shop work, which is quite high, also compare stresses, especially bending moments, the latter being particu-

FIG. 107*k*.—Single Plate Solid Portal for Bridge of Northern Pacific R. R. over Madison River, Montana. McClintic-Marshall Construction, Co., Rankin, Pa.

larly expensive to take care of. For the latter reason, the depth should be as great as the clearance will allow. See also Art. 102.

We may first divide as in riveted trusses into (*a*) *one*, or (*b*) *two plane* structures. In the former, the plane should be that of the neutral axis of the end post. For the latter, the

* There may often be considerable resisting power in this intermediate bracing, and a structure should not be condemned for lack of transverse strength without bearing this in mind.

two planes should be those of the top and bottom of end post. This is to be preferred for heavy bridges and where it is desired to use members of some length to carry compression.

Type (*a*). Here the sections are usually of two angles, one being occasionally specified for light tensile stresses. Except for large structures, the portal may be shop riveted, the connections being made by the usual single plate. A portal of this type is seen in Fig. 107*i*.

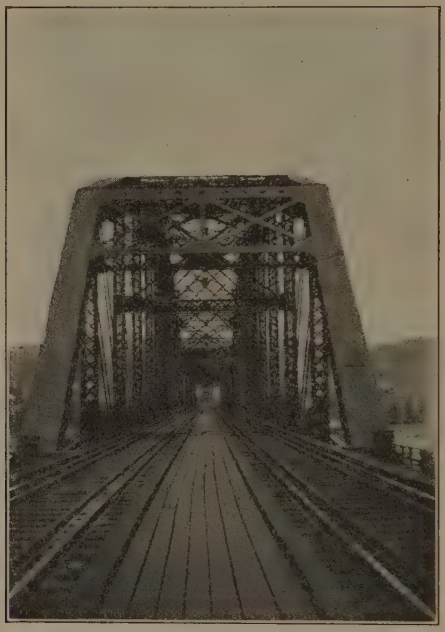


FIG. 107*l*.—Double Plate Portal to Bridge over Allegheny River at Aspinwall, Pa.; Brilliant Cut-off, Pennsylvania R. R.

Type (*b*) Fig. 107*j*. The customary section here is the four-angle I latticed together to form shapes capable of carrying compression. They fasten onto top chord by plates quite similar to those which connect top laterals.

Prob. 107. Assume Fig. 107*j* to represent portal for a Baltimore truss of 12 panels of 20 feet each and 36 feet high. What transverse load per lineal foot on top chords would it safely withstand? Use specifications of Art. 50.

Art. 108. Truss Bridges on Grades, Curves, and Skews

Grades

As in plate girders for grades met in practice, we may detail as though it were horizontal, perhaps multiplying actual span length by secant of the angle of inclination with the horizontal. No change will be required for the shoe, where, as is usually the case, the pin joint is employed.

Curves

Same methods may be employed as in plate girder bridges for curves except that elevation of shoes on one side to give proper superelevation should not be attempted.

Skews

The similarity between plate girders and truss bridges still

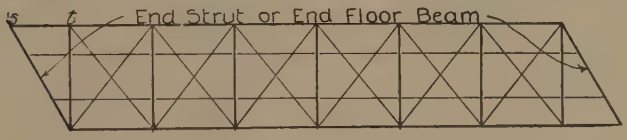


Fig. 108.
Floor Plan of Skew Truss Bridge.

continues. In a through structure with inclined end posts, we obtain a "skew portal" which involves unusual trouble in detailing. It will be treated in Vol. III. The usual arrangement is seen in Fig. 108.

Prob. 108. A railroad has passenger cars 60 feet long, 40 feet between center supports, 14 feet high, and 10 feet wide over all. What will be minimum spacing of trusses on a through single-track bridge 200 feet long with a vertical end post and a 3° curve with rails of standard gage superelevated for 60 miles per hour? Posts are 20″ wide.

Art. 109. Secondary Stresses

The stresses obtained in a truss with frictionless hinges by assuming that loads are applied centrally at panel points, act axially as a uniformly distributed stress over entire section. These are sometimes termed "primary stresses." Such ideal conditions are not realized in practice and the departure therefrom gives rise to other strains, the most important result being to produce flexural stresses. Whatever their nature, they are grouped under "secondary stresses." Causes may be classified as follows:

(1) *Weight of members*, Arts. 55 and 56.

(2) *Loads not at panel points*, which may be treated similarly to (1).

(3) *Eccentric application of load*, see Applied Mechanics.

(4) *Members not meeting at a point.* This is a bad detail and should be avoided except for light loads.

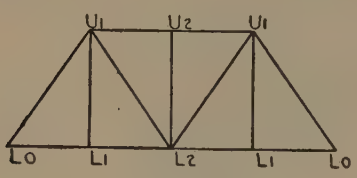


FIG. 109*a*.
Design.

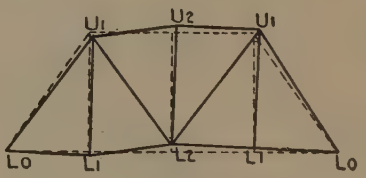


FIG. 109*b*.
Actual, U_1L_2 on the left too short.

Effect of a Single Inaccurate Piece.

(5) *Curved members.* These are expensive and inefficient, but if architectural considerations demand them, they should be treated like (3).

(6) *Misfits.* For example, the truss shown in Fig. 109*a*, will appear as in Fig. 109*b* if U_1L_2 were made a little short, thus introducing large bending stresses in top and bottom chords unless they be hinged at U_1, U_2, L_1, and L_2. Method of analysis is similar to that given in (8).

(7) *Temperature.* Both primary and secondary stresses may be caused in this way for statically indeterminate structures, see Vol. III. For a simple truss, they may be due to the

friction of the shoes, or to the deflection caused by the sun shining unequally on different parts. The latter case is handled as in (*8*).

(*8*) *Secondary Stresses due to Fixed Joints, Method of Computation.*

Any change of load produces alteration of stresses, deflection, and change in angles between panel points. With the perfect hinges assumed in computations, the members would adjust themselves to the new position. However, there is a great deal of friction in a pin joint. The angle between members meeting at a riveted joint must remain the same, this value being that at which the members were fastened together; usually under no load. For example, a riveted truss, ABC under a vertical load P at A, Fig. 109*c*, becomes $A'B'C'$ as shown dotted, the angle between the end tangents to the distorted

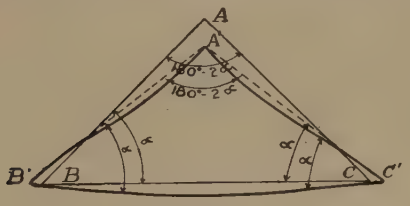


FIG. 109*c*.

A Deflected Triangular Truss.

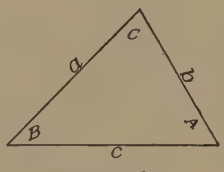


FIG. 109*d*.

Triangle of a Truss.

members remaining unchanged, causing them to bend to the form shown by the heavy lines. While the term "secondary stress" may be applied to any of the preceding, it particularly refers to the present cause, and this alone is considered in what follows. For a truss in which dimensions, loads, and sectional areas are given, we may divide computation into five parts:

(*A*) *Compute stresses.*

(*B*) *Obtain deformations.*

(*C*) *Determine alteration of angles between panel points.*

(*D*) *Find angle of distortion of joint.*

(*E*) *The resulting flexural stresses are the required values.*

The student should already be familiar with (*A*) and (*B*).

(*C*) *Alteration of Angles between Panel Points after Deflection.*

In the triangle ABC, Fig. 109*d*, let $\triangle a$, $\triangle b$, and $\triangle c$ represent the deformations of members a, b, and c respectively; plus if elongation and negative if a shortening. Let it be required to find the value of $\triangle A$, that is, the change in the angle A between the panel points.

The effect of the forces applied to these bars will be to alter the lengths. If now we take the new triangle with sides $a+\triangle a$, $b+\triangle b$, and $c+\triangle c$, and multiply each side by the fraction $\frac{c}{c+\triangle c}$, the angles remain unchanged, and the triangle has parts as shown in Fig. 109*e*. Recollecting that distortions are small compared with the length of the sides,

$$op=(a+\triangle a)\frac{c}{c+\triangle c}-a=(a+\triangle a)\frac{1}{1+\frac{\triangle c}{c}}-a$$

$$=(a+\triangle a)\left(1-\frac{\triangle c}{c}\right)-a=\triangle a-\triangle c\frac{a}{c}.$$

Similarly,

$$om=\triangle b-\triangle c\frac{b}{c}.$$

Constructing as in Williot's diagram, Fig. 109*f*, we may solve analytically as follows:

$$\tan\triangle A=\frac{mn}{b}=\frac{op\csc C-om\cot C}{b}$$

$$=\frac{\left(\triangle a-\triangle c\frac{a}{c}\right)\csc C-\left(\triangle b-\triangle c\frac{b}{c}\right)\cot C}{b}.$$

And in a like manner,

$$\tan\triangle B=\frac{\left(\triangle b-\triangle a\frac{b}{a}\right)\csc A-\left(\triangle c-\triangle a\frac{c}{a}\right)\cot A}{c};$$

$$\tan\triangle C=\frac{\left(\triangle c-\triangle b\frac{c}{b}\right)\csc B-\left(\triangle a-\triangle b\frac{a}{b}\right)\cot B}{a}.$$

Another method is to substitute in the above,

$$\triangle a = \frac{S_a a}{E}, \quad \triangle b = \frac{S_b b}{E}, \quad \text{and} \quad \triangle c = \frac{S_c c}{E},$$

where S_a, S_b, and S_c are the unit stresses in the members a, b, and c respectively, and E is the common modulus of elasticity. We then obtain,

$$E \tan \triangle A = (S_a - S_c)\frac{a}{b} \csc C + (S_c - S_b) \cot C;$$

$$E \tan \triangle B = (S_b - S_a)\frac{b}{c} \csc A + (S_a - S_c) \cot A;$$

$$E \tan \triangle C = (S_c - S_b)\frac{c}{a} \csc B + (S_b - S_a) \cot B,$$

which will in general be the quickest method. Or the problem may be solved by the usual trigonometrical formulæ for a triangle with three given sides, $a + \triangle a$, $b + \triangle b$, and $c + \triangle c$. Angles should then be computed to the nearest second.

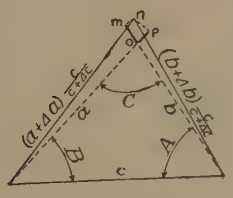


Fig. 109e.

Part of a Framework.

Distortions.

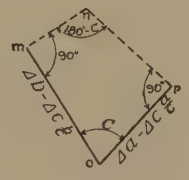


Fig. 109f.

Of a Joint.

(*D*) *Angle of Distortion of Joint.* This is at once the backbone and the difficulty of the whole problem. The forces which may act on a member of a deflected truss are:

Direct Stress, P at either end, Fig. 109g.

Shear V, positive when acting clockwise about right end.

Moments, M_0 and M_1, at ends, considered positive when acting clockwise as shown.

The usual treatment ignores the moment due to the direct load which is generally small. We have then for the equilibrium of the member as a whole,

$$M_0+M_1+Vl=0, \text{ hence } V=-\frac{M_0+M_1}{l}. \quad . \; . \; (1)$$

Equation of the elastic curve is $EI\frac{\delta^2y}{\delta x^2}=-M.$

Or,

$$EI\frac{\delta^2y}{\delta x^2}=-Mx=-M_0-Vx=-M_0+\frac{M_0+M_1}{l}x.$$

Integrating,

$$EI\frac{\delta y}{\delta x}=-M_0x+\frac{M_0+M_1}{2l}x^2+C_1.$$

When $x=0$, $\frac{\delta y}{\delta x}=-\alpha_1$, hence $C_1=-EI\alpha_1$.

$$x=l, \quad \frac{\delta y}{\delta x}=-\alpha_2, \quad \text{hence } EI(\alpha_2-\alpha_1)=\tfrac{1}{2}l(M_0-M_1). \; . \; (2)$$

Substituting,

$$EI\frac{\delta y}{\delta x}=-M_0x+\frac{M_0+M_1}{2l}x^2-EI\alpha_1.$$

Integrating again,

$$EIy=-\frac{M_0x^2}{2}+(M_0+M_1)\frac{x^3}{6l}-EI\alpha_1x+C_2.$$

When $x=0$, $y=0$, hence $C_2=0$.

$$x=l, \quad y=0, \quad \text{hence } \frac{M_1l}{2}=M_0l+3EI\alpha_1. \; . \; . \; . \; (3)$$

Eliminating M_1 from (2) and (3),

$$M_0=-\frac{2EI}{l}(2\alpha_1+\alpha_2). \; . \; . \; . \; . \; . \; . \; (4)$$

Let us designate each apex of the truss with a numeral. Then $l_{1,2}$, $E_{1,2}$, and $I_{1,2}$, represent respectively length, modulus of elasticity, and moment of inertia, of member between 1 and 2. Similarly $M_{1,2}$ signifies the moment at the 1 end of 1,2. $\theta_{1,2}$ and $\phi_{1,2}$, represent the inclination with the horizontal of line 1,2 before and after deflection respectively. β_1 designates the angle with the horizontal of a line on joint plate at 1 which was horizontal before deflection. There will be in general a different β for each joint. Clockwise rotation will be considered positive. $\alpha_{1,2}$, is the rotation around 1 in a clockwise direction of the tangent to the deflected 1,2 from the right line 1,2, the latter in its deformed position.

We now proceed as follows:

(*a*) Since, from the laws of equilibrium, the algebraic sum of the moments, M, about a point equals zero, we express this as a separate equation for each joint.

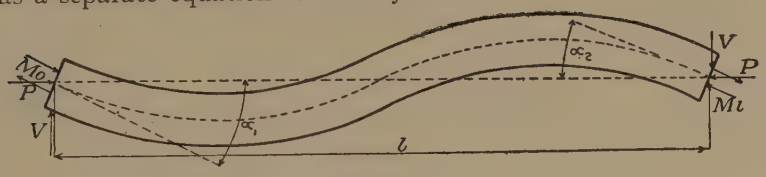


FIG. 109g.

Distortions of a Member.

(*b*) Substitute for each moment, M, its value from equation (4).

(*c*) In (*b*), state each α in terms of θ and ϕ known and β unknown, as follows:

$$\beta+\theta=\phi+\alpha \quad \text{or} \quad \alpha=\beta+\theta-\phi.$$

(*d*) Solve the equations for the values of the unknown β's. There will be as many equations with as many unknown quantities as there are joints.

(*E*) *Resulting Flexural Stresses.* With β's known, obtain in turn α_1 and α_2, M_0, M_1, and V. From these, flexural and shearing unit stresses may be obtained in the usual manner.

We will further explain method for obtaining secondary stresses due to fixed joints by computing the very simple example seen in Fig. 109*h*.

Example

Fig. 109*h*.

Everything in kips and inches. E = 30,000. Loads, 10 Top, 30 Bottom.

Member.	Stress. (*A*).	Area.	Length.	θ	Deformation.	
					(*B*).	(*C*). ϕ
10,1	125.0 C	19.80	300	126°-52′-11.6″	.0630−	126°-55′-29.7″
1,2	120.0 C	19.80	180	0-00-00	.0364−	0-03-13.1
2,3	135.0 C	19.80	180	0-00-00	.0409−	0-01-25.4
10,11	75.0 T	12.06	180	0-00-00	.0374+	0-04-46.7
11,12	75.0 T	12.06	180	0-00-00	.0374+	0-02-11.5
12,13	120.0 T	17.64	180	0-00-00	.0408+	0-01-46.0
1,11	30.0 T	8.92	240	90-00-00	.0269+	90-02-13.5
2,12	30.0 C	8.92	240	90-00-00	.0269−	90-01-10.1
3,13	10.0 C	8.92	240	90-00-00	.0090−	90-00-00.0
1,12	75.0 T	17.64	300	53-07-48.4	.0425+	53-09-51.7
2,13	25.0 T	8.92	300	53-07-48.4	.0280+	53-08-37.9

(*D*) *Determination of* β*'s.*

(*a*) At joint 10, $M_{10,1}+M_{10,11} = 0.$ (11)

1, $M_{1,10}+M_{1,11}+M_{1,12}+M_{1,2} = 0.$ (12)

2, $M_{2,1}+M_{2,12}+M_{2,13}+M_{2,3} = 0.$ (13)

11, $M_{11,10}+M_{11,1}+M_{11,12} = 0.$ (14)

12, $M_{12,11}+M_{12,1}+M_{12,2}+M_{12,13} = 0.$ (15)

At joints 3 and 13, equations are evidently identical from the symmetry of the trusses and the loads.

(*b*) From equation (4), $M = -\frac{2EI}{l}(2\alpha_1+\alpha_2)$. However, in writing our value for the moments, we may omit the constant, $-2E$. We then have, keeping everything in kips, inches and seconds. From

(11) $$\frac{I_{1,10}}{l_{1,10}}(2\alpha_{10,1}+\alpha_{1,10})+\frac{I_{10,11}}{l_{10,11}}(2\alpha_{10,11}+\alpha_{11,10})=0,$$

or $$2.084(2\alpha_{10,1}+\alpha_{1,10})+1.423(2\alpha_{10,11}+\alpha_{11,10})=0. \quad . \quad . \quad (21)$$

$$(12)\quad \frac{I_{1,10}}{l_{1,10}}(2\alpha_{1,10}+\alpha_{10,1})+\frac{I_{1,11}}{l_{1,11}}(2\alpha_{1,11}+\alpha_{11,1})+\frac{I_{1,12}}{l_{1,12}}(2\alpha_{1,12}+\alpha_{12,1})+\frac{I_{1,2}}{l_{1,2}}(2\alpha_{1,2}+\alpha_{2,1})=0,$$

or $$2.084(2\alpha_{1,10}+\alpha_{10,1})+0.558(2\alpha_{1,11}+\alpha_{11,1})+0.688(2\alpha_{1,12}+\alpha_{12,1})+3.473(2\alpha_{1,2}+\alpha_{2,1})=0. \quad (22)$$

$$(13)\quad \frac{I_{1,2}}{l_{1,2}}(2\alpha_{2,1}+\alpha_{1,2})+\frac{I_{2,12}}{l_{2,12}}(2\alpha_{2,12}+\alpha_{12,2})+\frac{I_{2,13}}{l_{2,13}}(2\alpha_{2,13}+\alpha_{13,2})+\frac{I_{2,3}}{l_{2,3}}(2\alpha_{2,3}+\alpha_{3,2})=0,$$

or $$3.473(2\alpha_{2,1}+\alpha_{1,2})+0.558(2\alpha_{2,12}+\alpha_{12,2})+0.446(2\alpha_{2,13}+\alpha_{13,2})+3.473(2\alpha_{2,3}+\alpha_{3,2})=0. \quad (23)$$

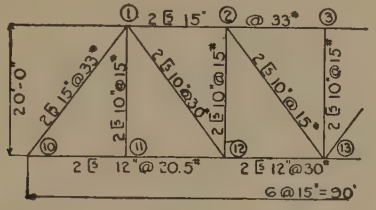


FIG. 109*h*.—Riveted Truss of Example.

$$(14)\quad \frac{I_{10,11}}{l_{10,11}}(2\alpha_{11,10}+\alpha_{10,11})+\frac{I_{1,11}}{l_{1,11}}(2\alpha_{11,1}+\alpha_{1,11})+\frac{I_{11,12}}{l_{11,12}}(2\alpha_{11,12}+\alpha_{12,11})=0.$$

$$1.423(2\alpha_{11,10}+\alpha_{10,11})+0.558(2\alpha_{11,1}+\alpha_{1,11})+1.423(2\alpha_{11,12}+\alpha_{12,11})=0. \quad (24)$$

$$(15)\quad \frac{I_{11,12}}{l_{11,12}}(2\alpha_{12,11}+\alpha_{11,12})+\frac{I_{1,12}}{l_{1,12}}(2\alpha_{12,1}+\alpha_{1,12})+\frac{I_{2,12}}{l_{2,12}}(2\alpha_{12,2}+\alpha_{2,12})+\frac{I_{12,13}}{l_{12,13}}(2\alpha_{12,13}+\alpha_{13,12})=0.$$

$$1.423(2\alpha_{12,11}+\alpha_{11,12})+0.688(2\alpha_{12,1}+\alpha_{1,12})+0.558(2\alpha_{12,2}+\alpha_{2,12})+1.797(2\alpha_{12,13}+\alpha_{13,12})=0. \quad (25)$$

(*c*) Now $\alpha = \beta + \theta - \phi$, hence

$\alpha_{1,2} = \beta_1 - 193.1$. $\alpha_{1,10} = \beta_1 - 198.1$. $\alpha_{1,11} = \beta_1 - 133.5$.

$\alpha_{1,12} = \beta_1 - 123.3$.

$\alpha_{2,1} = \beta_2 - 193.1$. $\alpha_{2,3} = \beta_2 - 85.4$. $\alpha_{2,12} = \beta_2 - 70.1$.

$\alpha_{2,13} = \beta_2 - 49.5$.

$\alpha_{3,2} = -85.4$. $\alpha_{3,13} = 0$, since $\beta_3 = 0$ from symmetry.

$\alpha_{10,1} = \beta_{10} - 198.1$. $\alpha_{10,11} = \beta_{10} - 286.7$.

$\alpha_{11,1} = \beta_{11} - 133.5$ $\alpha_{11,10} = \beta_{11} - 286.7$. $\alpha_{11,12} = \beta_{11} - 131.5$.

$\alpha_{12,1} = \beta_{12} - 123.3$. $\alpha_{12,2} = \beta_{12} - 70.1$. $\alpha_{12,11} = \beta_{12} - 131.5$.

$\alpha_{12,13} = \beta_{12} - 106.0$.

$\alpha_{13,2} = -49.5$. $\alpha_{13,3} = 0$. $\alpha_{13,12} = -106.0$.

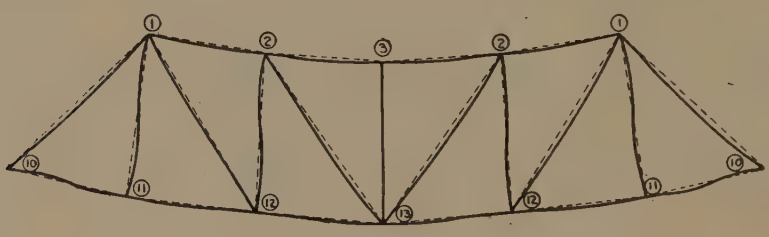


FIG. 109*i*.—Distorted Truss.

Substituting above values, we obtain,

From (21) $7.014\beta_{10} + 2.084\beta_1 + 1.423\beta_{11} = 2462.3$. (31)

(22) $13.606\beta_1 + 3.473\beta_2 + 2.084\beta_{10} + 0.558\beta_{11} + 0.688\beta_{12} = 3728.7$. (32)

(23) $15.900\beta_2 + 3.473\beta_1 + 0.558\beta_{12} = 3085.5$. (33)

(24) $6.808\beta_{11} + 0.558\beta_1 + 1.423\beta_{10} + 1.423\beta_{12} = 2008.8$ (34)

(25) $8.932\beta_{12} + 0.688\beta_1 + 0.558\beta_2 + 1.423\beta_{11} = 1504.5$ (35)

Solving,

$\beta_1 = 182.6$. $\beta_2 = 150.4$. $\beta_{10} = 255.4$. $\beta_{11} = 203.1$. $\beta_{12} = 112.6$.

(*E*) *Resulting Flexural Stresses.*

$\alpha_{1,2} = -10.5$. $\alpha_{1,10} = -15.5$. $\alpha_{1,11} = +49.1$. $\alpha_{1,12} = +59.3$.
$\alpha_{2,1} = -42.7$. $\alpha_{2,3} = +65.0$. $\alpha_{2,12} = +80.3$. $\alpha_{2,13} = +100.9$.
$\alpha_{3,2} = -85.4$. $\alpha_{3,13} = 0.0$.
$\alpha_{10,1} = +57.3$. $\alpha_{10,11} = -31.3$.
$\alpha_{11,1} = +69.6$. $\alpha_{11,10} = -83.6$. $\alpha_{11,12} = +71.6$.
$\alpha_{12,1} = -10.7$. $\alpha_{12,2} = +42.5$. $\alpha_{12,11} = -18.9$. $\alpha_{12,13} = +6.6$.
$\alpha_{13,2} = -49.5$. $\alpha_{13,3} = 0.0$. $\alpha_{13,12} = -106.0$.

Now $M_{mn} = -\frac{2EI}{l}(2\alpha_{mn} + \alpha_{nm})$, where α is in radians. Taking E as 30,000, and 1 second equals .000004848 radians, formula (4) becomes,

$$M_{mn} = -0.2909\,\frac{I}{l}(2\alpha_{mn} + \alpha_{nm}),$$

where α is in seconds. Also, (1) may be stated

$$V_{mn} = -\frac{M_{mn} + M_{nm}}{l_{mn}}.$$

We may now tabulate final results as follows, everything in kips and inches:

Moments, *M*.				Shears, *V*.	Flexural Unit Stress.	Per Cent of Primary Stress.
1,10	−15.9	10,1	− 60.3	+.25	0.72	11.4
1,2	+64.4	2,1	+ 96.9	−.90	1.16	19.1
2,3	−45.1	3,2	+106.9	−.34	1.28	18.8
10,11	+60.3	11,10	+ 82.2	−.78	1.92	31.0
11,12	−51.5	12,11	− 14.0	+.36	1.20	19.4
12,13	+48.5	13,12	+107.4	−.87	2.00	29.4
1,11	−27.3	11,1	− 30.6	+.24	1.14	34.0
2,12	−33.0	12,2	− 27.0	+.25	1.23	36.6
3,13	0.0	13,3	0.0	.00	0.00	0.0
1,12	−21.6	12,1	− 7.6	+.10	0.52	12.3
2,13	−19.1	13,2	+ 0.2	+.06	0.71	25.3

Other Methods

The solution just presented is due to Mohr. There are several other methods, but we have space for but a brief statement of two of them.*

(1) *Manderla's Method.* This takes account of the eccentricity of the axial load on the deformed truss member, thus usually increasing moment for compression and decreasing it for tension. Evidently its principal effect will be on long thin members. As columns in well-designed trusses are made quite stiff and its effect on tension members is to lessen the secondary stresses, this method is seldom advisable, especially as it is quite complicated.†

(2) *Influence Method.* Here each load is considered in turn, and its effect ascertained. To obtain the largest secondary stress, live load is placed at only those points which will produce moments of the same kind. However, we usually wish maximum *sum of primary plus secondary*, and this may be taken with sufficient accuracy for most cases to occur under full live load.‡

Designs to Reduce Secondary Stresses

Under the most favorable conditions, the chords must bend to the general curve of deflection. This means exactly as in a solid beam,

$$S_u = \frac{P_u}{A_u}, \quad \frac{h+d_1}{h}, \quad \text{Fig. 109}j.$$

$$S_l = \frac{P_l}{A_l}, \quad \frac{h+d_2}{h},$$

where S_u and S_l represent stresses at uppermost and lower-

* For remaining methods see "Secondary Stresses in Bridge Trusses," by Grim.

† For proof and formula see same volume.

‡ For numerous examples, see "Modern Framed Structures," Part II, by Johnson, Bryan, and Turneaure.

most fibers respectively. The secondary stress is the difference between the above values and the axial P_u/A_u and P_l/A_l equals

$$\frac{Pd}{Ah}.$$

This may be considered an ideal value for the chords which the engineer will seek to attain but never reach. Due to the restraining action of the web members, values will run much higher.

To reduce secondary stresses,

(*A*) Avoid trusses, which, like the Whipple and lattice, have two or more theoretically independent systems; for loads on one only produce very large secondary stresses, Fig. 109*k*.

(*B*) The secondary verticals in Baltimore and Pennsylvania trusses and the verticals in a Warren truss are also objectionable, since they cause, in deflecting, abrupt changes of direction in the chords, Fig. 109*l*.

(*C*) In a quite similar manner, "bracing" or "collision" struts, Art. 103, may be equally detrimental.

(*D*) Aim to make curve of deflection an approximate circle with vertical members radial. This means uniform stress in chords and low stresses elsewhere.

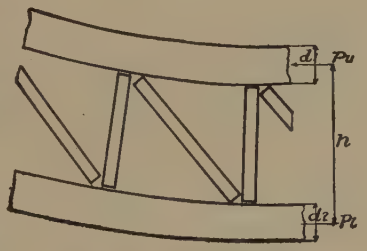


Fig. 109*j*.—Deflected Truss.

(*E*) Keep moments of inertia as small as possible. It is particularly desirable that those of the web members shall be low.

(*F*) Employ a depth not less than the economic.

(*G*) Try to avoid large moments of inertia in more than two members at a joint.

(*H*) A pin joint at certain points may help a great deal. However, it must be remembered that there is always more or less friction which prevents the theoretic assumption of no moment at a pin from being strictly true.

It will be noted that some of the above rules interfere with what is otherwise good design. In this case the engineer must use his judgment with the probability that the lesser weight should be attached to the secondary stresses.

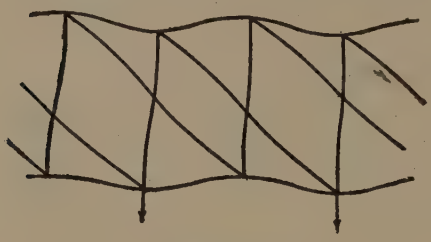


FIG. 109*k*.
Whipple.

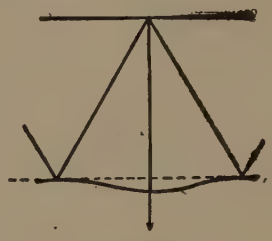


FIG. 109*l*.
Warren with Sub-verticals.

Deflected Trusses.

REFERENCES

"Secondary Stresses in Bridge Trusses," by Grim.

"Theory and Practice of Modern Framed Structures," Part II, by Johnson, Bryan, and Turneaure.

Engineering News, Vol. 66, p. 397; Vol. 68, p. 438; Vol. 71, p. 619.

Journal of Western Society of Engineers, Vol. 17, p. 101.

Prob. 109. In the truss of the above example, consider that the member 11,12 on each side is $\frac{1}{8}''$ short, and compute secondary stresses due to this alone.

Art. 110. Loads and Specifications for Truss Bridges

Loads due to own weight may be obtained from Art. 102; specifications for railroad truss bridges from Art. 50; specifications for highway bridges from Arts. 91 and 92; we will note here but one small difference for truss bridges—the wind should be supposed to act on *each* projection of the truss.

We cannot too strongly condemn the careless design of many highway truss bridges. The loads may be light and the impact less than for railroad structures, but there is an equal danger and equal need for high-grade engineering.

For purposes of instruction, we have used specifications

which vary considerably. While there are none which contain unsound or unsafe clauses, they are intended to represent all types. For example, Art. 111 exemplifies light work and high allowable stresses such as might be employed in competitive work. On the other hand, Art. 93 gives expensive but substantial construction. As an engineer, the author inclines very decidedly towards the latter. A great deal inevitably depends upon the circumstances of the case, and must be left to the decision of the designer in charge.

Art. 111. Lattice Girders—Example

A small riveted truss with chords parallel or nearly so is sometimes termed a "lattice girder." They may be employed for short highway bridges to lessen difficulties of transportation, or for elevated structures to decrease the obstruction to the light. See Figs. 98*h* and 98*k*.

As an example of their use, let it be required to design a light bridge 7 feet wide in the clear and 100 feet long for a footway. It is to be supported on steel bents at either end.

The live loads are per sq.ft.—100 for floors and stringers; 85 for floor beams; and 70 for trusses. For wind, 30, giving estimated pressures of 75 and 150 lbs. per lin.ft. on top and bottom chords respectively. Notice that live and wind will not occur simultaneously.

We will allow the following unit stresses in lbs. per sq.in.:

Tension or flexure..... 15,000

Compression..... $15{,}000 - 80\frac{l}{\rho}$.

Shear on shop rivets and net sections of webs..... 12,000
Shear on field rivets and bolts..... 9,000
Bearing on rivets and bolts.....Twice above values for shear
Flexure of hard pine..... 1,000

Stresses 20% greater are allowed for members carrying wind.

Minimum thickness, $\frac{5}{16}''$; minimum leg, $2\frac{1}{2}''$. Rivets in general are $\frac{3}{4}''$ dia.

(*a*) *Wooden Floor*

If we use 2″ ($1\frac{3}{4}$″ actual thickness) plank supported every two feet, it will be found to give ample strength with a margin for wear. For the hard pine stringers, $w = (100+10)2 = 220$ lbs. per lin.ft. Hence, $bd^2 = \frac{3}{4}\frac{wl^2}{S} = .0138l^2$, when everything is expressed in inches.

		Required.	Furnished.	
For 14 panel lengths,	$l = 85.7''$,	$bd^2 = 101$,	use $3'' \times 6''$,	$bd^2 = 108$
13	92.3	117	$4'' \times 6''$	144
12	100.0	138	$4'' \times 6''$	144
11	109	164	$3'' \times 8''$	192
10	120	198	$4'' \times 8''$	256

Of which the third seems preferable. However, we shall use the last with 2″×8″ for side stringers, because it will economize on shop work by making fewer pieces.

(*b*) *Floor Beams*

8′–0″ long, 10′–0″ center to center.

Load per lin.ft., 850+120=970, say 1000.
Maximum moment = 96,000 in.lbs.
Section modulus required, 96,000/15,000 = 6.4.
Use one I, 6″ @ 14.75#, $I/c = 8.0$.

(*c*) *Panel Loads per Truss*

Estimated dead load of flooring and stringers...	420 lbs.
Estimated dead load of floor beams.....	60 lbs.
Estimated dead weight of the truss per lin.ft. from formula (5) of Art. 102 is 65#. Multiplying by panel length and adding 40% for bracing which is proportionately larger than usual, we have.....	910
Total which we shall place $\frac{1}{3}$ top and $\frac{2}{3}$ bottom.	1390, say 1400

Live, 70×4.0×10 on bottom chord.....		2800
Wind, 75×10 on top chord	Considered...	750
Wind, 150×10 on bottom chord	as dead.....	1500

(*d*) *Economical Depth*

From Art. 102, (3), $h = 0.42\sqrt{pL} = 13.3$ feet.

Assumptions made in deducing formula make result too large; also the fact that the minimum size often governs will tend the same way. We will try 9 feet, leaving 7′ 2″ in the clear, about the minimum allowable for foot traffic.

(*e*) *Truss Members*

Shop rivet, value $\frac{3}{4}'' \times \frac{1}{2} \times 24{,}000 = 9{,}000$ lbs.
Minimum number, 2.

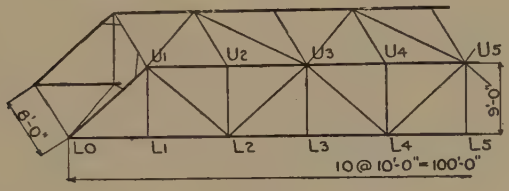


FIG. 111*a*.

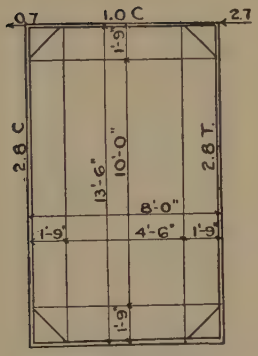


FIG. 111*b*.—Portal.

FIG. 111*c*.—End Post.

Table below is based on $\frac{1}{2}''$ connection plates which renders strength in bearing equal to 85% of that in double shear. Where there is a reversal of stress and it is apparent which one governs, only such is given.

All values in kips and inches

Member.	Stresses.							Use.	Avail. Area.	ρ	Allow Unit Stress.	Allow Total Stress.	Shop Rivets.
	Dead.	Live.		Wind.		Total.							
		T	C	T	C	T	C						
U_1U_2	12.4 C	....	24.8	3.3	3.3	....	37.2	2 L's 4″×3″×$\frac{3}{8}$″	4.96	1.26	7.40	36.8	5
U_2U_3	12.4 C	....	24.8	5.6	5.6	....	37.2						
U_3U_4	18.7 C	....	37.4	7.0	7.0	....	56.1	2 L's 4″×3″×$\frac{5}{8}$″	7.96	1.23	7.20	57.5	7
U_4U_5	18.7 C	....	37.4	7.4	7.4	....	56.1						
L_0L_1	7.0 T	14.0	....	12.6	12.6	21.0	5.6		....	....	....	....	3
L_1L_2	7.0 T	14.0	....	19.2	19.2	21.0	12.2		....	....	....	....	3
L_2L_3	16.3 T	32.6	....	23.9	23.9	48.9	7.6	2 L's 4″×3″×$\frac{3}{8}$″	4.24	....	15.00	63.6	6
L_3L_4	16.3 T	32.6	....	26.7	26.7	48.9	10.4		....	....	....	....	6
L_4L_5	19.5 T	39.0	....	27.6	27.6	58.5	8.1		....	....	....	....	7
Verticals	0.5 C 1.0 T	2.8	....	....	....	3.8	0.5	1 L 3″×3″×$\frac{5}{16}$″	....	....	.....	....	2
U_1L_0	9.5 C	....	19.0	....	....	....	28.5	.....	See (f) below				4
U_1L_2	7.3 T	15.1	0.4	....	....	22.4	....	2 L's 3$\frac{1}{2}$″×2$\frac{1}{2}$″×$\frac{5}{16}$″	3.02	....	15.0	45.3	3
U_3L_2	5.3 C	1.3	11.7	....	....	....	17.0	2 L's 3$\frac{1}{2}$″×2$\frac{1}{2}$″×$\frac{1}{2}$	5.50	1.09	3.1	17.0	2
U_3L_4	3.1 T	8.8	2.5	....	....	11.9	....	2 L's 2$\frac{1}{2}$″×2$\frac{1}{2}$″×$\frac{5}{16}$″	2.40	....	15.0	36.0	2
U_5L_4	1.0 C	4.2	6.3	....	....	3.2	7.3	2 L's 3$\frac{1}{2}$″×2$\frac{1}{2}$″×$\frac{5}{16}$″	3.56	1.12	3.5	12.5	2

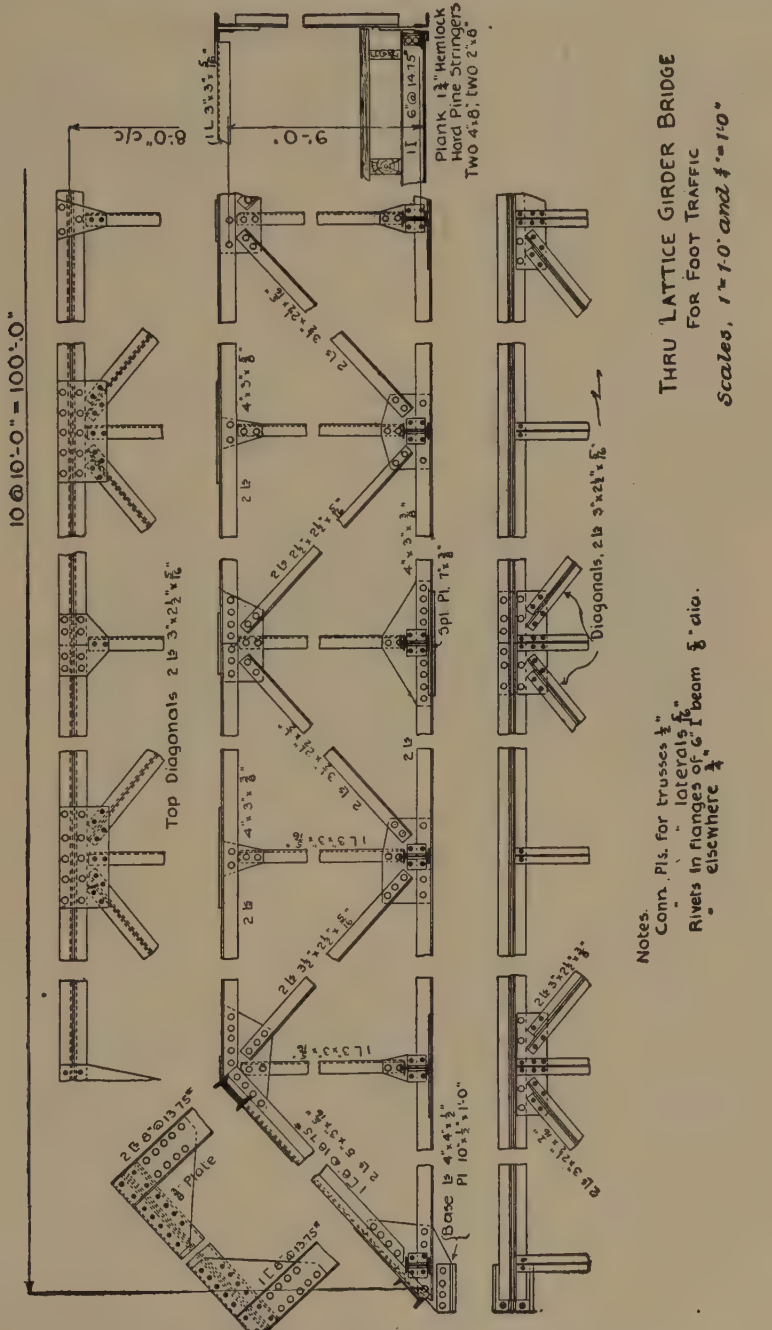


FIG. 111*d*.—General Drawing of Lattice Girder.

(*f*) *End Portal*, Fig. 111*b*.

The maximum bending moment in U_1L_0, or U_0U_0, = $1.7 \times 60 = 102$ kip in. See Art. 99.

For U_0U_0, we will employ 2 [’s 8″ @ 13.75#, $\frac{3}{8}$″ apart, $\rho = 0.98$″.

Allowable unit stress with wind 8.6.

$$\text{Actual stress} = \frac{P}{A} + \frac{Mc}{I} = \frac{1.7}{8.1} + \frac{102.0}{18.0} = 5.8. \quad \text{O.K.}$$

At the bottom, the floor beam will care for direct compression and a single channel 8″ @ 13.75# will carry moment safely.

For U_1L_0, we decided after several trials to employ section seen in Fig. 111*c*, consisting of two L's, $5'' \times 3'' \times \frac{5}{16}''$ and one channel 8″ @ 18.75#.

For this we computed the following quantities:

Total area, 10.31 sq.in., $\frac{I}{c}$ transversely to truss is 12.7, $\rho = 1.43$″, unsupported length decreased to 11 feet on account of fixed ends, allowable unit stress, 9.2 kips per sq.in., actual unit stress,

$$\frac{12.3}{10.3} + \frac{102.0}{12.7} = 9.2 \text{ kips per sq.in.} \quad \text{O.K.}$$

(*g*) *Laterals*

For the diagonals, two L's, $3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$, will carry in compression 7.5 kips. This will suffice except in first and second panels of lower laterals, where thicknesses must be $\frac{7}{16}$ and $\frac{3}{8}$″ respectively. For the upper struts, one L $3'' \times 3'' \times \frac{5}{16}''$ will do.

Prob. 111. How much material will be saved or lost by making depth in above problem 10 feet?

Art. 112. Pony Trusses—Example

The writer does not believe that there is often a legitimate excuse for the pony truss. Unless figured and designed to carry the wind pressure and to stiffen the top chord by means of the posts, it is dangerous; if this is done, it will not be economical. Where transportation facilities permit, use a plate girder; otherwise, a truss bridge deep enough for the overhead bracing.

Other than the need for additional stiffness, this type does not differ from other riveted trusses. It usually has two plate

FIG. 112*a*.—Pony Truss Bridge over N. Branch Chicago River, Addison Ave., Chicago, Ill.

connections and is largely confined to highways. However, it has been employed on railroads.

EXAMPLE

Let it be required to design a city bridge of 120 feet span with a heavy traffic. It is to have a roadway 30 feet in the clear and a sidewalk on both sides of 7 feet net width. We will use plank floors, steel joists, and pony trusses. Loading 120, rivets $\frac{3}{4}$ dia. of soft steel; other material, medium steel; all open hearth. Use specifications of Art. 92.

This being a comparatively wide bridge, formula (7*a*) of Art. 102 should be employed rather than (8*a*). This gives an eco-

nomical panel length of 16.8 feet, when we take w as 200 *×48 =9600, d to be 40 and n 14. We will divide into 10 panels of 12 feet each, thus enabling us to utilize the floor of problem in Art. 93. This is also better suited to the depth chosen below.

Trusses

Economical depth $= 0.42\sqrt{12 \times 120} = 16$ feet. (Art. 102.)

It would be better to make this enough for a system of upper laterals, say 20 feet, but as we have set out to illustrate the pony truss, it is now preferable to reduce somewhat, as it lessens the cost of stiffening the top chord. Let us try 12 feet.

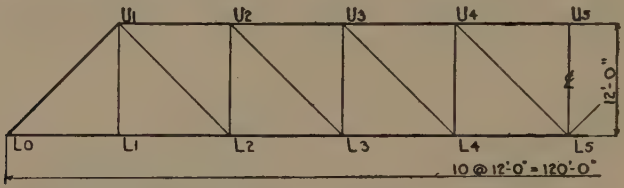
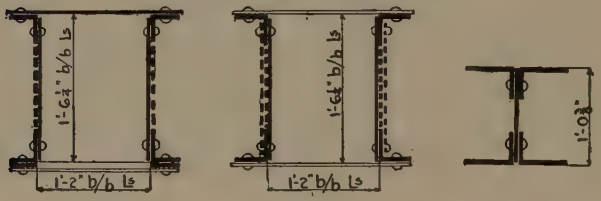


Fig. 112*b*.—Truss of Problem.



Top Chord. Bottom Chord. Posts and Diagonals.

Fig. 112*c*.—Sections of Truss Members.

Rivets are ¾″ dia. Conn. Pls. at Shoe ⅝″; elsewhere ½″. Everything in kips and inches.

Dead panel load, Art. 93, 12.0 kips.

From Art. 102, (5), using 120 for L, 15,000 for S, 200 *×48 =9600 for w and 12 for p, we get weight per foot per truss to be 511 lbs. Allowing 150 lbs. per lin.ft. for extras,† makes total dead panel load 20 kips, which we will divide 4 top and 16 bottom.

Uniform live panel load, Art. 93, 36.0 kips.

Concentrations taken as 11,000 lbs. and handled like locomotive loads.

* Estimated from table of shears and moments in girder, Art. 93.

† This type is not economical of material and will weigh more in consequence.

Member.	Stresses.				Use.	Available Area.	p	Allowable.	
	Dead.	Live Unif.	Live Conc.	Total.				Unit.	Total.
U_1L_0	127C	229C	77C	433C	1 Top Pl. 22″×½″ 2 Web Pls. 18″×⅜″ 2 Top L's 3½″×3½″×½″ 2 Bott. L's 3½″×3½″×$\frac{7}{16}$″	35.20	7.1	12.4	437C
U_1U_2	160C	288C	94C	542C	1 Top Pl. 22″×½″ 2 Web Pls. 18″×$\frac{7}{16}$″ 2 Top L's 3½″×3½″×½″ 2 Bott. L's 3½″×3½″×⅝″	41.21	7.2	13.2	545C
U_2U_3	210C	378C	124C	712C	U_1U_2+2 Pls. 11″×⅜″	54.96	6.5	13.0	715C
U_3U_4	240C	432C	142C	814C	1 Top Pl. 22″×½″ 2 Web Pls. 18″×⅝″ 2 Web Pls. 11″×⅜″ 4 L's 3½″×3½″×⅝″	63.17	6.4	13.0	824C
U_4U_5	250C	450C	147C	847C	U_3U_4+2 Bars 3½″×⅛″	65.90	6.7	13.1	865C
L_0L_1	90T	162T	54T	306T	2 Pls. 18″×⅜″ 4 L's 4″×4″×⅜″	20.36	.....	15.0	306T
L_1L_2									
L_2L_3	160T	288T	94T	542T	L_0L_1+ 2 Pls. 10″×⅜″ 2 Pls. 16″×⅜″	36.67	.....	15.0	550T
L_3L_4	210T	378T	124T	712T	2 Pls. 18″×$\frac{9}{16}$″ 4 L's 4″×4″×$\frac{9}{16}$″ 2 Pls. 10″×$\frac{9}{16}$″ 2 Pls. 16″×$\frac{9}{16}$″	54.93	.....	15.0	822T
L_4L_5	240T	432T	142T	814T					
U_1L_1	16T	36T	17T	69T					
U_2L_2	54C	101C	37C	192C					
		11T	4T						
U_3L_3	34C	76C	28C	138C	See below				
		22T	8T						
U_4L_4	14C	54C	20C	88C					
		36T	13T	35T					
U_5L_5	4C	0	0	4C					
U_1L_2	99T	183T	64T	346T	1 Pl. 12″×⅝″ 4 L's 6″×4″×$\frac{9}{16}$″	23.25	.....	15.0	349T
		5C	2C						
U_2L_3	71T	142T	52T	265T	1 Pl. 12″×$\frac{7}{16}$″ 4 L's 6″×4″×$\frac{7}{16}$″	18.12	.....	15.0	272T
		15C	6C						
U_3L_4	42T	107T	40T	189T	1 Pl. 12″×$\frac{7}{16}$″ 4 L's 4″×3″×⅜″	13.04	.....	15.0	196T
		31C	12C						
U_4L_5	14T	76T	28T	118T	4 L's 4″×3″×⅝″	8.56	.....	15.0	128T
		51C	19C	56C	⅝″ apart	9.92	2.03	6.0	60C

Computation of Posts

These we will assume to carry in addition to the stresses given above, one-tenth of stress in top chord, considered as a short compression member. However, since the latter has a varying section, we will consider each post to bear its share, one-tenth of one-tenth, of average section at that panel point, that is,

$$\text{At } U_1 = 0.01(35.2+41.2)\tfrac{1}{2}\cdot 15.0 = 5.73 \text{ kips}$$
$$U_2 = 0.01(41.2+55.0)\tfrac{1}{2}\cdot 15.0 = 7.22 \text{ kips}$$
$$U_3 = 0.01(55.0+63.2)\tfrac{1}{2}\cdot 15.0 = 8.86 \text{ kips}$$
$$U_4 = 0.01(63.2+65.9)\tfrac{1}{2}\cdot 15.0 = 9.70 \text{ kips}$$
$$U_5 = 0.01 \quad 65.9 \quad 15.0 = 9.90 \text{ kips}$$

The floor beam, about three feet deep, will be placed just above the bottom chord. We shall consider the point of application of the above load to be $12.0-3.0-1.0=8.0$ feet or 96 inches from the critical section. Also let us compute the following quantities for the verticals, see Fig. 112*b*, keeping everything in kips and inches:

		Area.	ρ	Allow Comp.	I/c.
(1)	1 Pl. $12''\times\frac{1}{2}''$ 4 L's $6''\times4''\times\frac{1}{2}''$	25.00	2.57	10.0	98
(2)	1 Pl. $12''\times\frac{5}{8}''$ 4 L's $6''\times4''\times\frac{5}{8}''$	30.94	2.62	10.0	120

Next we prepare tabulation for posts:

Member.	Total Stress.	Use.	Moment.	I/c.	Unit Stresses.			
					Flexural.	Direct.	Total.	Allowable
U_1L_1	69 T	(1)	550	98.0	±5.6	$+2.8$	-2.8 $+8.4$	10.00 15.00
U_2L_2	192 C	(2)	695	120.0	±5.8	-6.2	-12.0	on net 10.00
U_3L_3	138 C	(2)	850	120.0	±7.1	-4.4	-11.5	10.00
U_4L_4	88 C	(2)	930	120.0	±7.8	-2.8	-10.6	10.00
U_5L_5	4 C	(2)	950	120.0	±7.9	-0.1	-8.0	10.00

As the requirements were rather severe, this will be taken as sufficient, the stronger posts being assumed to assist those overburdened.

Wind Bracing

For the wind load we shall take the rather high value of 500 lbs. dead per lineal foot, in part to take care of an inclined wind pressure against a wide floor.

Maximum stress in end panel is 33.8 kips T. Area required, 33.8/15 is 2.25 sq.in. Use one L, $3'' \times 3'' \times \frac{1}{2}''$ with a net of 2.31 sq.in. and 12 $\frac{3}{4}''$ field rivets.

Maximum stress in third panel is 18.8 kips T. Area required, 18.8/15 is 1.25 sq.in. Use one L, $3'' \times 3'' \times \frac{5}{16}''$ with a net of 1.51 sq.in. and 7 field rivets, $\frac{3}{4}''$ dia.

Maximum stress in end strut is 15 kips C. Use four L's, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{16}''$ latticed. $A = 8.36$, $\rho = 1.70$, unsupported length, 16 feet, allowable stress, 4.90 kips per sq.in.

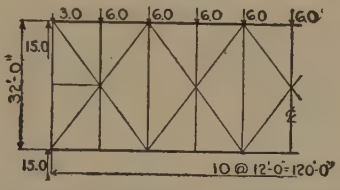


Fig. 112*d*.—Arrangement of Bottom Laterals.

Shoe, Maximum reaction, concentrations not considered, 314 kips;

$$\text{Bearing required on } 5'' \text{ pin} = \frac{314}{5 \times 20} = 3\tfrac{1}{8}'', 3\tfrac{3}{4}'' \text{ furnished.}$$

$$\text{Lineal inches, } 4\tfrac{3}{4}'' \text{ rollers required} = \frac{314}{0.4 \times 4.75} = 165.$$

$$\text{Bearing required on masonry} = \frac{314}{0.4} = 785 \text{ sq.in.}$$

Art. 113. Computation of a Railroad Double-track Deck Bridge

Let it be required to design a Warren truss bridge of 160 feet span for a double-track railroad. Use specifications of Art. 50 except that live load is to be 5000 lbs. per lin.ft. plus two concentrations of 100,000 lbs. each, 3 panel lengths apart,

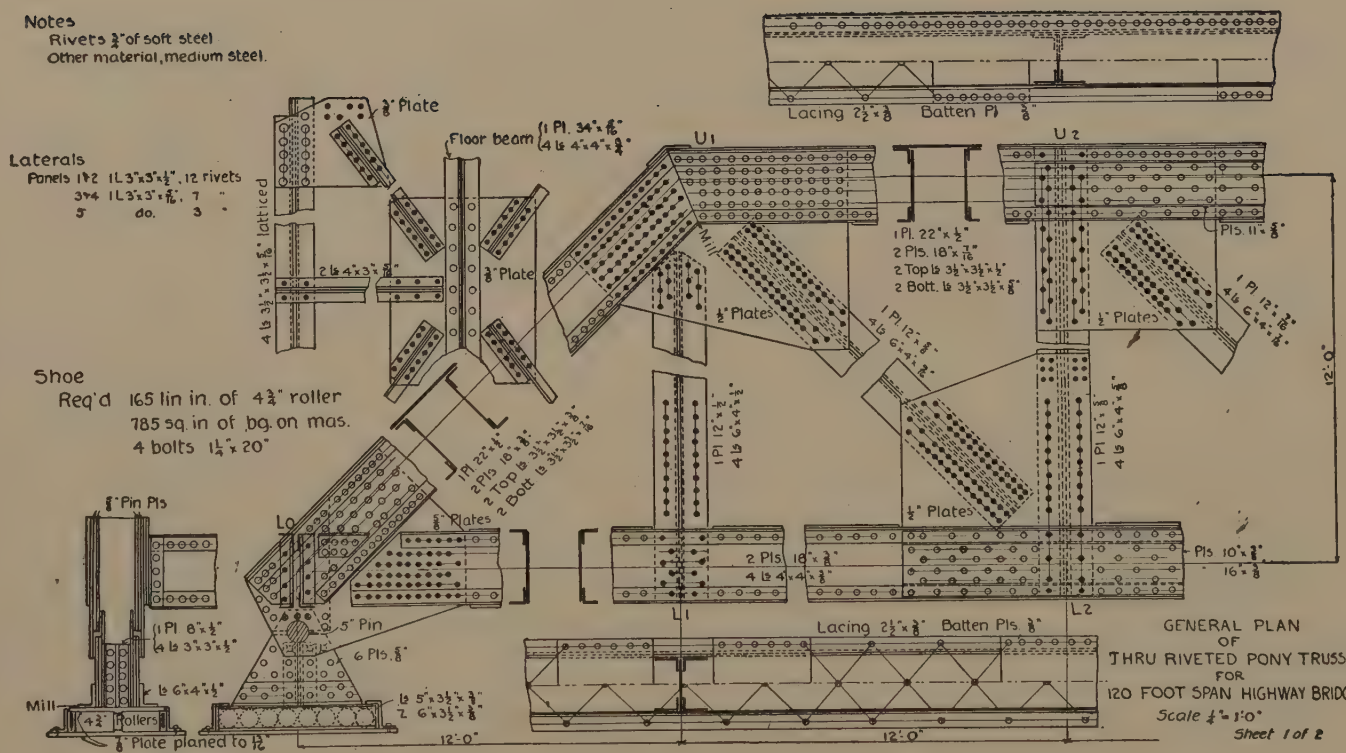

Notes
Rivets ⅞" of soft steel
Other material, medium steel.
Laterals
Panels 1-2 1 L 3"×3"×½", 12 rivets
3-4 1 L 3"×3"×⅜", 7 "
5 do. 3 "
⅜" Plate
Floor beam
1 Pl. 34"×⅝"
4 Ls 4"×4"×⅝"
U1
U2
Lacing 2½"×⅜" Batten Pl ⅜"
1 Pl. 22"×½"
2 Pls. 18"×⅞"
2 Top Ls 3½"×3½"×½"
2 Bott. Ls 3½"×3½"×⅝"
Pls. 11"×⅝"
½" Plates
1 Pl 12"×⅝"
4 Ls 6"×4"×⅝"
12'-0"
Shoe
Req'd 165 lin in. of 4¾" roller
785 sq in of bg. on mas.
4 bolts 1¼"×20"
⅝" Pin Pls
L0
L1
L2
2 Pls 18"×⅞"
4 Ls 4"×4"×⅝"
Pls 10"×⅝"
16"×⅜"
1 Pl 8"×½"
4 Ls 3½"×3½"×½"
5" Pin
6 Pls. ⅝"
Ls 6"×4"×½"
Ls 5"×3½"×⅜"
2 " 6"×3½"×⅝"
Mill
4½" Rollers
⅝" Plate planed to ½"
12'-0"
Lacing 2½"×⅜" Batten Pls ⅜"
GENERAL PLAN
OF
THRU RIVETED PONY TRUSSES
FOR
120 FOOT SPAN HIGHWAY BRIDGE
Scale ¼"=1'-0"
Sheet 1 of 2

FIG. 1128.

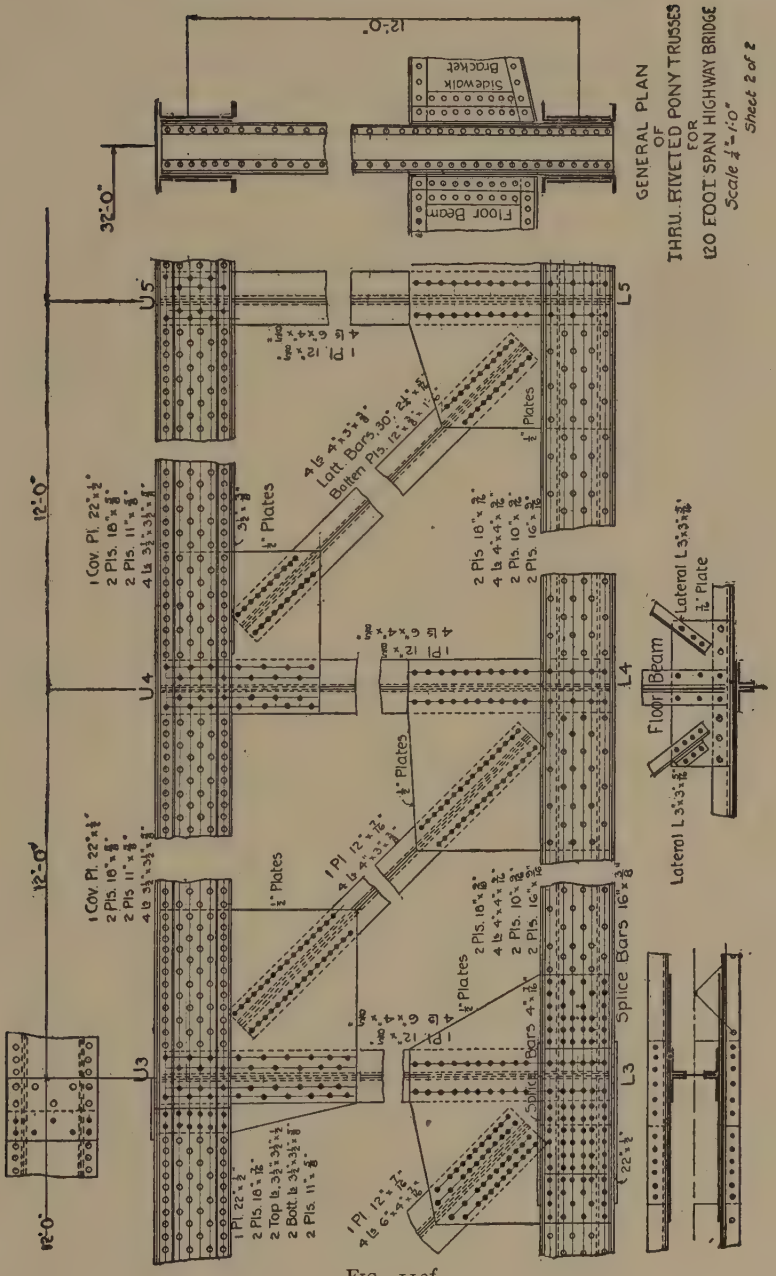

GENERAL PLAN
OF
THRU. RIVETED PONY TRUSSES
FOR
120 FOOT SPAN HIGHWAY BRIDGE
Scale 1/4" = 1'-0"
Sheet 2 of 2
Sidewalk Bracket
Floor Beam
12'-0"
32'-0"
U5
U4
U3
L5
L4
L3
Lateral L 3"x3"x3/16"
Lateral L 3"x3"x3/16"
Floor Beam
Splice Bars 16"x3/8"
Splice Bars 4"x7/16"
Latt. Bars 30"x2 1/4"x5/16"
Batten Pls. 12"x3/8"x1'-6"
1 Cov. Pl. 22"x1/2"
2 Pls. 18"x5/8"
2 Pls. 11"x5/8"
4 Ls 3 1/2"x3 1/2"x5/8"
1 Pl. 12"x7/16"
4 Ls 6"x4"x7/16"
1 Pl. 22"x1/2"
2 Pls. 18"x7/16"
2 Top Ls. 3 1/2"x3 1/2"x1/2"
2 Bott. Ls 3 1/2"x3 1/2"x5/8"
2 Pls. 11"x5/8"
2 Pls. 18"x7/16"
4 Ls 4"x4"x7/16"
2 Pls. 10"x7/16"
2 Pls. 16"x9/16"
1/2" Plates
22"x1/2"

FIG. 112*f*.

both per track. Wind loads, 800 lbs. per lin.ft. on top chord and 400 on bottom. Rivets are $\frac{7}{8}''$ dia. of soft steel, other material medium steel.

General Dimensions

Center to center trusses is $7.0+13.0=20.0$.

Economical panel length, Art. 102 (7), $w=18{,}000$, $n=4$, $S=10{,}000$, is 17.9, use 8 @ 20.0 feet.

Economical depth, Art. 102 (3), is 23.8, use 25.0 feet.

Stringer

Stringer will be a built I and we will employ the *intermediate* method for flanges and *approximate* for rivet spacing.

Estimated dead loads per lin.ft. per stringer: track, 200 lbs.; stringer 120 lbs.*

Maximum shear, 78.2 kips, M,† 1.02. Use $36''\times\frac{3}{8}''$ web plate.

Maximum moment, 4693 kip in., M, 1.02. Use flange angles $6''\times6''\times\frac{5}{8}''$. $6''\times4''$ with the latter vertical will not hold enough rivets.

Horizontal shear for rivets is $78{,}200/33.0=2370$ lbs. per lin.in. Assumed vertical shear per lin.in., 700 lbs. Resultant, 2470 lbs. Spacing of $\frac{7}{8}''$ rivets is $1.64''$ at end. At center the same procedure gives $3.30''$.

Stringer will weigh about 150 lbs. per lin.ft.

Floor Beam

Estimated weight of floor beam, 5000 lbs.

Stringers spaced 7.0 feet from nearest truss.

Floor beam reactions: dead, 7.0; live, 100.0 kips.

Maximum shear, 110 kips, M is 1.05. Use $60''\times\frac{3}{8}''$ web.

Maximum moment 9140 kip in., M is 1.04. Use four L's, $6''\times6''\times\frac{3}{8}''$ and two cover pls. $14''\times\frac{3}{8}''\times11.0$ feet long.

Rivet spacing between stringers and trusses $=\dfrac{4.14\cdot58.5}{110}$

$=2.20''$.

* It will weigh a little more but not enough to affect our computations.

† $M=1+\frac{1}{2}\dfrac{\text{Minimum}}{\text{Maximum}}$.

Panel Loads, in Kips

Dead, floor, 14.0; dead, truss, 30. Total 44.0, 32.0 upper and 12.0 lower chord. Uniform live, 100, on top chord. Concentrated live, 2 of 100 kips, 3 panels apart. Wind, 16.0 top, and 8.0 bottom, considered as on one side.

Design of Trusses

Wind loads will not have sufficient value to affect computations according to specifications. Use two-plate connections of $\frac{3}{4}''$ material. $20''$ is about right for the clear distance between plates, this being such as to make radii of gyration in top chord about equal. We will make it $20\frac{5}{8}''$ in the clear.

Tension Members

Fig. 113*a*.

Everything in kips and inches

Member.	Max. Stress.	*M.*	Allowable Stress.	Area Req'd.	Use.	Gross Area.	Holes Deducted.	Net Area.
L_0L_1 } L_1L_2 }	513T	1.12	11.2	45.8	2 Pls. $30''\times\frac{1}{2}''$	30.00	10	25.00
					4 L's $6''\times6''\times\frac{1}{2}''$	23.00	4	21.00
					Totals	53.00		46.00
L_2L_3 } L_3L_4 }	1074T	1.13	11.3	95.0	2 Pls. $30''\times\frac{1}{2}$	30.00	10	25.00
					4 L's $6''\times6''\times\frac{1}{2}''$	23.00	4	21.00
					2 Pls. $18''\times\frac{1}{2}''$	18.00	6	15.00
					4 Pls. $28''\times\frac{3}{8}''$	42.00	20	34.50
					Totals	113.00		95.50
U_1L_1 } U_3L_3 }	122T	1.11	11.1	11.0	2 ['s 15" @ 33 #	19.80	6	17.40
U_1L_2	621T	1.08	10.8	57.5	2 Pls. $30''\times\frac{11}{16}''$	41.25	10	34.38
					4 L's $6''\times6''\times\frac{5}{8}''$	28.44	4	25.94
					Totals	69.69		60.32
U_3L_4*	268T	0.79	7.9	33.8	2 Pls. $20''\times\frac{9}{16}''$	22.50	8	18.00
					4 L's $4''\times4''\times\frac{5}{8}$	18.48	4	15.98
					Totals	40.98		33.98

* Must stand compression also, but tension governs.

Compression Members

Member.	Max. Stress.	M.	Use.	Area.	ρ.	Allowable.	
						Unit.	Total.
U_3L_2	437C	1.00	2 Pls. 24″×⅝″ 4 L's 6″×6″×⅝″	30.00 28.44			
			Totals.....	58.44	8.9	7.4	433
U_1L_0	822C	1.12	2 Pls. 30″×½″ 4 L's 6″×6″×½″ 2 Pls. 18″×½″ 2 Pls. 28″×½″	30.00 23.00 18.00 28.00			
			Totals.....	99.00	9.4	8.4	832
U_0L_0	166C	1.05	2 ['s 15″ @ 45 #	26.48	5.32	7.0	185
U_2L_2 U_4L_4	232C	1.07	2 ['s 15″ @ 55 #	32.36	5.16	7.0	226

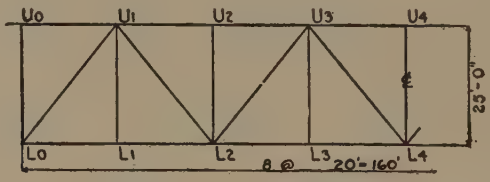
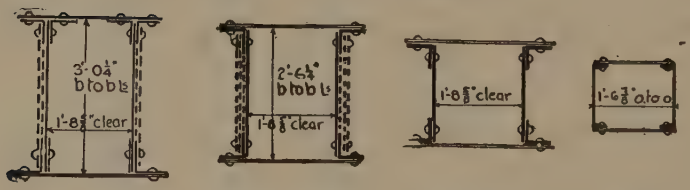


Fig. 113*a*.—Truss of Example.



Top Chord. Bottom Chord. Diagonals. Verticals.

Fig. 113*b*.—Sections of Truss Members.

Members in Flexure Only

Moment in U_0U_1, taken same as stringer, 4693 kip in., M is 1.02, I/c required is 460. Use one cover plate, 36″×½″; two web plates, 36″×⅜″; four L's, 6″×6″×⅜″; $I/c=510$, which will allow for one open hole in each bottom angle and side plate.

Members Carrying Flexure and Compression

In U_1U_2 and U_2U_3, maximum moment is 4693 kip in. Direct compression is 870 kips and M is 1.12. Use

1 cover plate, $36'' \times \frac{1}{2}''$.
4 top Ls, $6'' \times 6'' \times \frac{9}{16}''$.
2 bottom Ls, $8'' \times 8'' \times \frac{9}{16}''$.
2 web plates, $36'' \times \frac{9}{16}''$.
2 side plates, $22'' \times \frac{9}{16}''$.
2 vertical bottom plates, $8'' \times \frac{3}{8}''$.

Total area, 132.4 sq.in. $I_{c.g.}$ about a vertical axis, 16,310 in.^{4}, about horizontal, 22,940; $\rho = 11.1''$. Allowable unit stress, 9.75 kips per sq.in. I/c for tension, 1080; for compression, 1470. There is no tension and maximum compression is,

$$\frac{870}{132.4} + \frac{4693}{1470} = 9.73 \text{ kips per sq.in.}$$

The bottom plates need not extend the full length. Without them, area is 126.4 sq.in., I/c for compression fiber is 1420 in.^{3}, and allowable unit stress is unchanged. Letting x equal allowable moment in kip in.,

$$\frac{870}{126.4} + \frac{x}{1420} = 9.75, \quad x = 4080.$$

Assuming now the parabolic curve for the moments, we have that distance of cut off point from center is,

$$\sqrt{\frac{4690 - 4080}{4690}} \cdot 10 = 3.6 \text{ feet},$$

and the bottom plates should be about 8 feet long.

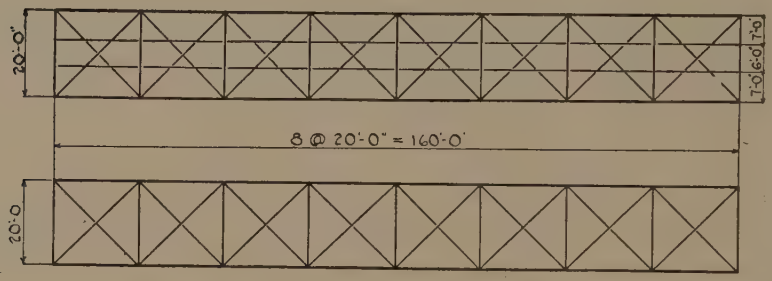
Maximum moment in U_3U_4 is 4693 kip in. as before. Direct compression, 1121 kips, and M, 1.12. Use same section as before with two additional side plates, $34'' \times \frac{1}{2}''$, giving a total area of 166.4 sq.in., a radius of gyration of $11.2''$, an I/c, vertically, of 1640, an actual stress of 9.60 kips per sq.in. as

compared with 9.75 allowable. In the same manner as before it may be computed that 5 feet of $8'' \times \frac{3}{8}''$ are needed.

It must not be supposed that these sections can be named offhand to come correct. Often, especially with beginners, several revisions are necessary. Note that for similarly shaped members, allowable unit stresses vary little. This is often of use in making the first estimate of section required and in experienced hands will bring close results.

Design of Laterals. Fig. 113*c*

Diagonals are figured to carry either tension or compression. *Top Laterals.* Unsupported length, 120″. Use two L's,



Upper Laterals. Lower Laterals.

Fig. 113*c*.

$5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, with the 5″ legs vertical and riveted in contact. Gross area is 6.10 sq.in., $\rho = 1.34''$, and allowable unit stress is $10{,}000\left(1 - .006\frac{120}{1.34}\right) = 4640$ lbs. per sq.in. Allowable total stress is 28.4 kips. The maximum stress is 39.5 kips and 80% of this as provided in the specifications is 31.7. Obviously $\frac{7}{16}''$ material will be needed in end panels and $\frac{3}{8}''$ will do elsewhere.

Bottom Laterals. Unsupported length is 170″. Use four L's, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ latticed to make them $\frac{3}{4}''$ apart. Area is 9.92 sq.in., $\rho = 1.75''$. Allowable unit stress is 4.20 kips per sq.in. Evidently this section may be used throughout. Struts will need to be designed for radius of gyration only since their

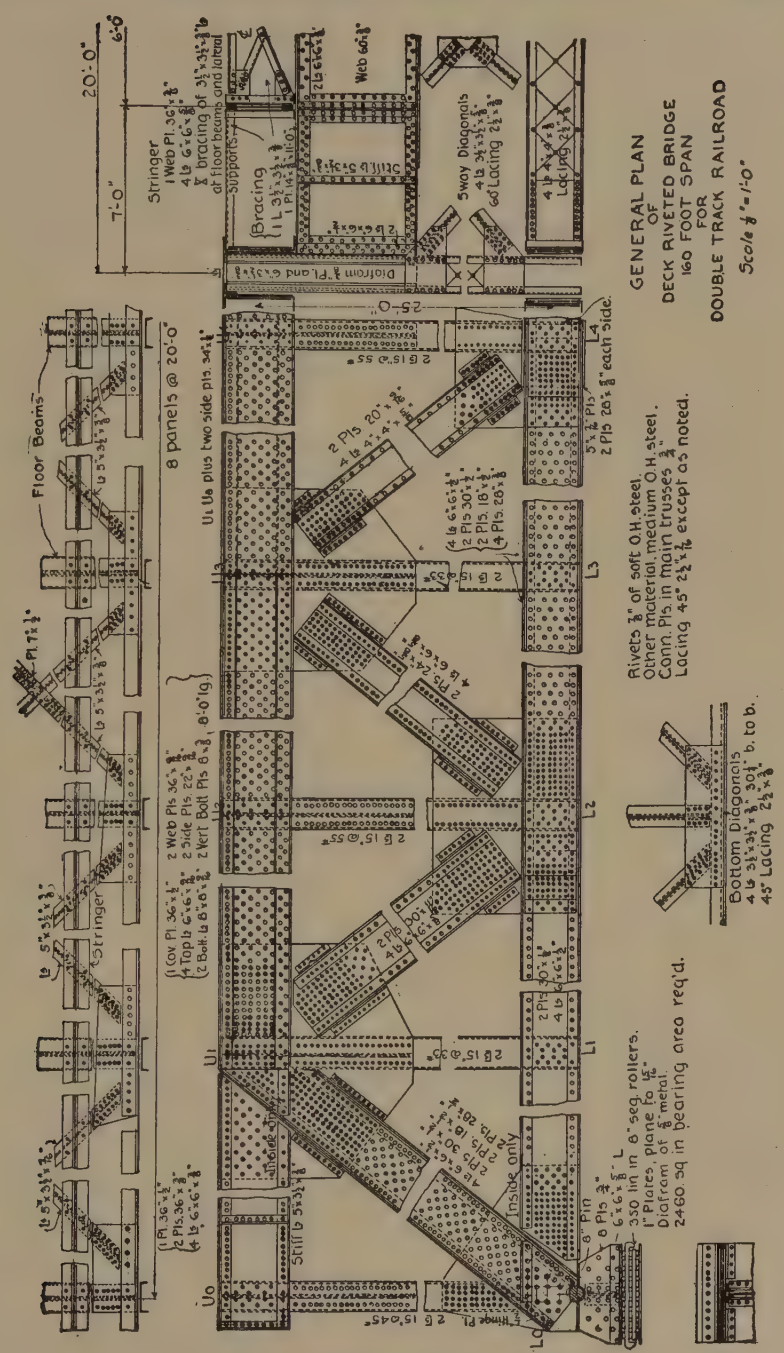


FIG. 113*d*.—General Drawing of Bridge.

stress is quite small. Length, 240″, required radius of gyration, 1.92. Use four L's, 4″×4″×$\frac{3}{8}$″, latticed to be $\frac{3}{4}$″ apart, for which $\rho = 1.95$.

Sway Laterals. Maximum stress in end sways is 80% of 51.2 = 41.0 T or C. Unsupported length is 192″. Use same section as for bottom laterals which have here an allowable stress of 3.4 kips per sq.in. Plainly, thickness must be increased to $\frac{1}{2}$″. For remaining sways with their nominal stresses, $\frac{3}{8}$″ material will suffice.

Design of Shoes

Maximum reaction, 738 kips. $M = 1.11$.

$$\text{Bearing required on 8″ pin} = \frac{738}{14.8 \times 8} = 6.25″.$$

$$\text{Bearing required on masonry} = \frac{738}{0.3} = 2460 \text{ sq.in.}$$

$$\text{Bearing required on 8″ rollers} = \frac{738}{8 \times 0.3} = 307 \text{ lin.in.}$$

CHAPTER IX

SIMPLE PIN-CONNECTED BRIDGES

Art. 114. General

While we have thus subdivided our simple truss bridges, it is not at all uncommon to find structures where there are both pin and riveted joints. It certainly seems better engineering to use each in those locations for which it is best adapted, even though they occur in the same truss, unless shop or field work is thereby rendered too expensive.

This chapter will be shorter than the one preceding. The subjects are of equal importance, but matters relating to them both were discussed in our last chapter. These we shall refer to, only noting differences therefrom. The principal points of dissimilarity are due to:

(*A*) The use of pins instead of rivets.

(*B*) They are usually longer spans for reasons stated in Art. 101.

(*C*) There is less stiffness at joints and hence it is not in keeping to attempt as much rigidity elsewhere. Further, the proportion of dead load increases with the span and reversals are less frequent and vibrations not as troublesome.

For references, see Art. 101.

Art. 115. Forms and Sections of Trusses

As already noted, pin joints are used in the longer spans and for such we find the Baltimore, Parker and Pennsylvania the typical trusses. The pin-connected Warren truss is seldom seen. The Pratt still remains a favorite for short spans, although somewhat crowded by tendency towards bending the upper chord and towards riveted trusses, more particularly for railroad bridges.

EYEBARS. (See also Art. 43.)

The most important change is the use of eyebars for tension members. Except at ends, these are plain rectangular bars, whose depth divided by the breadth may vary from 4 to 8. A smaller value would have high stresses under own weight and would take a great deal of room at joint and cause large moment on pin. A greater value is difficult to fabricate and would be likely to allow bar to buckle under own weight or under bearing from the pin. The thicknesses vary by sixteenths of inches, with a maximum of 2″. The usual depths are,

2, $2\frac{1}{2}$, 3, $3\frac{1}{2}$, 4, 5, 6, 7, 8, 10, 12, 14, and 16 inches.

FIG. 115*a*.—Through Pin-connected Pratt Truss Bridge over Heart River on Northern Pacific Railway. McClintic-Marshall Construction Co., Rankin, Pa.

The end may be thickened to afford the proper bearing area at pin, but this is not usual nor advisable, the ordinary method being to make diameter of pin at least $\frac{3}{4}$ of the depth of the bar. If made less, this detail will limit strength since bearing unit stress is usually given as $\frac{4}{3}$ times tensile. The head is a circle whose diameter is such as to give a net area 30 to 50% in excess of that of the body of the bar. The corners at *a*, Fig. 115*d*, are eased by curves of a radius which varies according to the manufacturer. For each depth of bar, there are two

FIG. 115*b*.—Through Pin-connected Parker Truss Bridge over Allegheny River at Aspinwall, Pa., Brilliant Cut-off, Pennsylvania R. R.

FIG. 115*c*.—Through Pin-connected Pennsylvania Truss Bridge at the Federal Street Crossing of the Allegheny River.

or three sizes of head, for each of which there is a maximum size of pin. For example, American Bridge Co. standards give for a 6″ bar, two heads: one of 14″ diameter for a maximum pin of $5\frac{3}{4}$″, providing an efficiency of $(14.0-5.75)/6=1.37$; also one of $14\frac{3}{4}$″ diameter for a $6\frac{1}{2}$″ pin with the same efficiency.

Except for small stresses an even number should be specified. This is to prevent a twisting of the joint. While three may be used in one direction, it will be likely to cause a moment undesirably high.

Eyebars may be counterbraced by methods seen in Figs. 115*e* and 115*f*. The latter for light thrusts may be reduced to two angles with lacing. The loss of effective area and additional cost for shop work and material are such that built

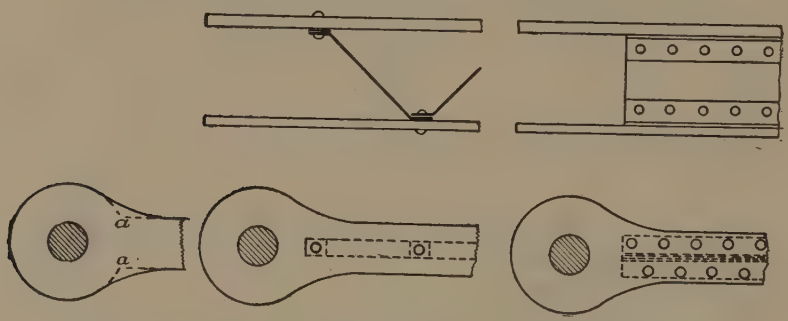


FIG. 115*d*. Eyebar Head.

FIG. 115*e*. FIG. 115*f*. Counterbraced Eyebars.

members with capacity for both tension and compression are preferable. An objection to the latter is the awkward appearance. The two end members of bottom chord are often stiffened. This is to carry the compression due to wind, traction, or the rusting of the expansion joints. Sometimes entire bottom chord is made stiff to render truss rigid or take care of stresses due to cantilever erection. For the diagonals of low trusses, counters are cheaper than counterbracing, but the latter is more rigid and growing in favor for railroad bridges. Counters are usually made adjustable, Art. 43. Length of shorter member should not then be less than $6\frac{1}{2}$ feet.

Verticals which carry tension only, for example, the hip

vertical in a through Pratt truss, are sometimes made of eye-bars. If the floor beam is attached to truss below the lower chord, construction seen in Fig. 59*f* may be employed. However, this is very poorly designed to resist bending stresses due to eccentricity of wind bracing. Method of Fig. 115*g* may be used if floor beam fastens above bottom chord, but it is expensive from the standpoint of the shop. Both are lacking in rigidity, and will not be seen in best designs of recent date. If possible, dimensions should be such as to keep floor beams alike. The best way, the only one for railroads or first-class highway bridges, is to make the hip vertical like intermediate posts; or, in other words, a compression member of a net area sufficient to carry its tension.

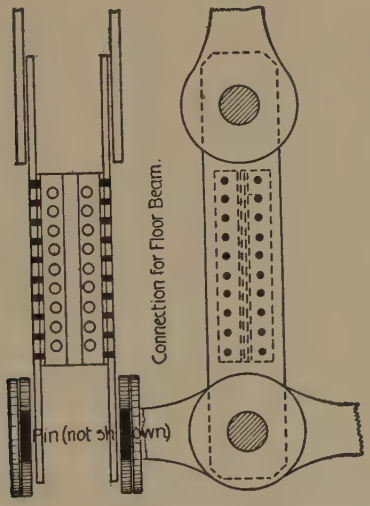


FIG. 115*g*.—Hip Vertical and Bottom Chord.

For all compression web members with pin joints, the two-channel column with lacing is best. Webs are placed parallel to plane of truss. Flanges may be turned in or out, the former giving a more compact section, and usually less cutting of the flanges. On the other hand, rivets are somewhat harder to drive. The distance in the clear should not be less than $4\frac{1}{2}''$.

For the upper chord, including end post of through bridges, the two channel and one cover plate section is typical. Figs.

115*h*, 115*i*, 115*j*, and 115*k* show common forms. The first can be used only for short spans, more particularly highway structures. The third and fourth are employed when the second would be unwieldy, that is, for the heavier spans.

There are three practical points here which must be borne in mind:

(1) The clear width inside must be sufficient to contain post and eyebars with which it connects.

(2) The cover plate (at the top) tends to keep the neutral axis pretty well up in the section. Now the proper location of the pin is just enough below the neutral axis so that eccentricity of stress balances moment due to own weight. Even with this, unless pains are taken to prevent, the eyebar heads are likely to interfere with top of cover plate. The latter may be slotted or the former planed, but both processes are expensive and weakening and should be considered a last resort. A better method is to bring down the center of gravity. This may be done by using heavier angles at the bottom, by tacking on extra plates, Figs. 115*l* and 115*m*,* or additional angles,

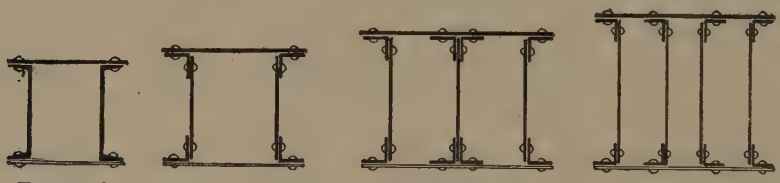


FIG. 115*h*. FIG. 115*i*. FIG. 115*j*. FIG. 115*k*.

Common Chord Sections.

Fig. 115*n*. The second adds to the shop work, while the third and fourth may interfere with clearance of connecting members.

(3) Sections must be such as to splice together readily and yet carry the variations of stress with least waste of material. It is desirable that the neutral axis be kept as nearly as possible in the same location, otherwise pin must be placed in a mean position, thus causing secondary stresses where not at the theoretic location.

The vertical back to back of angles is made about $\frac{1}{4}''$ greater than the webs; the horizontal back to back is made enough for

* These additional plates are termed "balance bars." Unless holes are located along median line, punching will distort them.

clearance; then the top plate must at least cover the angles. It is usual to make the latter the minimum allowable thickness throughout; to keep back to back and gage to gage of top and bottom angles alike. Thickness of vertical webs may be increased if there is room at joints; angles can be made thicker; or outside or inside plates added. The first outside web plate is made equal to the depth of the inside web minus the two adjacent legs of the angle. If still other webs are to be added, outside plate should have a thickness equal to the heavier angle. Next outside plate must be large enough to give the necessary edge distances and small enough to clear the fillets of the angles.

FIG. 115*l*. FIG. 115*m*. FIG. 115*n*.

Strengthened Bottom Flanges of Top Chords.

Prob. 115. A through Pratt highway bridge has 10 panels of 18 feet each. The maximum stresses in the top chord, beginning at the hip joint, are, 384, 504, 576 and 600 kips. Design in medium steel, using allowable stresses of Art. 92. Space required in the clear, 15″.

Art. 116. Splices

Eyebars are not spliced.

The need for splicing posts and counterbraced diagonals will seldom arise; if it should, they may be treated like top chord.

The entire top chord, including the end post in through bridges, is made, in a way, continuous. There are four different methods by which it may be spliced:

(1) *The Pin Splice*

Here all stresses are transmitted through the pin, rendering joint much like the hinge assumed in theory. The hinge plates shown are sometimes omitted; while they add to the difficulty of erection, they provide a safeguard against accidental displacement. This splice usually occurs at a meeting place with

other members. In the body of a column, it is likely to weaken the piece a great deal. A marked disadvantage of this type is the material necessary to properly distribute stress over section after its concentration at pin. d is usually $\frac{1}{4}''$ and adjoining edges are faced. This distance should be enough so that under extreme deflection, surfaces do not touch. Present tendency is to omit hinge plates.

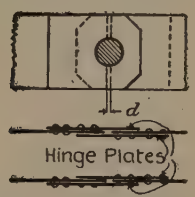


FIG. 116*a*.—Pin Splice.

(2) *Pin with Bearing*

This differs from preceding in having bearing surfaces in contact, thus transmitting the stresses, the pin carrying increment only. Needless to say, abutting edges must be faced. This should be done in a manner to give complete contact under maximum load. The usual method is to give the bridge a camber and then mill as though it were level. This seems like an efficient and economical joint, but there is a possibility of a dangerous concentration of stress due to the inaccuracy of shop work and non-fulfilment of theoretical assumptions in deflection of truss.

(3) *Riveted*

Here the necessary material and rivets are provided for splicing joint and ends do not bear. It is seldom used and we will not consider it further.

(4) *Riveted with Bearing*

Section is faced and is spliced by not less than two rows of rivets, preferably on all four sides. It will look right when spliced for 50% of load.

In either (3) or (4), splice should be located as near a panel point as is practical. If erection of top chord begins at middle, field splices should be on abutment side of panel point.

FIG. 116*b*.
Method of Splicing Top Chord.

At the hip of a through truss, there should be a joint of type (1) or (2), preferably the former. Where the upper chord is straight, it is usually field spliced every other panel length by

a joint of type (4). To avoid the awkward entering joint in erection, we may field rivet plates on both sides, after bolting for shipment. Another method is seen diagrammatically in Fig. 116*b*. Intermediate shop splices may be required by change in section. However, designer should strive to avoid the additional shop work if possible.

For bent upper chord, types (1), (2) or (4) may be used at angles with a preference for (1) or (4). Splice in either event must be made at intersection point.

The reader has no doubt noticed that the construction of pin bridges allows of considerable secondary stresses.

Art. 117. The Computation of Pin Joints

The pin has already been described in Art. 46 and pin joints briefly in Art. 60. A pin may be considered as a large bolt with threads and nut on either end. The preference for the two channel section is now explained by the ease with which connection can be made by means of pin holes bored through its webs. However, they must often be reinforced by pin plates to prevent bearing values from reaching too high an amount.

The diameter of the pin for simple trusses may vary from 3 to 10″, the actual determination of the size being explained in Art. 120. A round must be ordered enough larger so that it may be turned, shop practice varying in this respect.

With the Lomas nut now commonly used, the distance between shoulders of pin should be made equal to the thickness of the metal to be gripped. In computing this, add $\frac{1}{16}''$ to the nominal thickness of each eyebar, and $\frac{1}{4}''$ to the nominal size of each built member. This should be increased for bars over 8″ deep. Note that the height of the rivet must be added if they will interfere. Computing as above, thickest built member plus outside eyebars gives desired distance.

Decreasing size of pin tends to lessen material and shop costs; it is wise to do this as much as possible without increasing costs other ways. Let us therefore keep shear and moment low, more particularly the latter. Bear in mind the following points:

(1) Keep joint as compact as is possible.

(2) Place each member as close as practicable to that from which it receives its stress.

(3) If one member transmits stress into two others, place *it* between *them*.

To exemplify way in which these requirements are best filled, we give a sketch showing arrangement at joints. A more elaborate plan giving some details and dimensions is called a " packing plan," see Figs. 121*d* and 122*d*.

If bars are out of parallel to truss by over 1% of its length, they must be bent. Therefore, it is better to keep within this limit. Washer fillers, Art. 46, can be put in where necessary

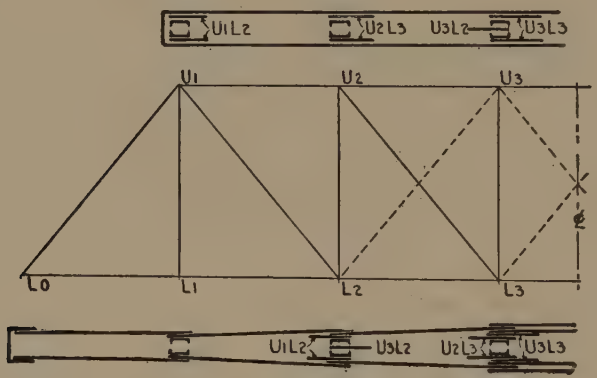


FIG. 117*a*.—Arrangement of Members at Pin Joints of Pratt Truss.

to prevent bars working out of position. For example, two should be used at each end of the counter, U_3L_2, Fig. 117*a*. Two eyebars for the same member should be kept at least an inch apart to allow of painting and inspection.

The thickness of the pin plates is determined in the usual way. That is,

$$\text{Total required thickness} = \frac{\text{Stress transferred}}{\text{Dia. pin} \times \text{allowable bearing unit stress}},$$

the units being in pounds and inches. Not less than the required thickness is furnished by one or more plates. Next these plates are assumed to carry an amount proportional

to their contact. Enough rivets must be provided, not only to drop the stress out of the pin plates, but to safely transfer it into all parts of the section.

In tension members, there should be an excess of net area of not less than 25% around the pin hole to provide for the unequal distribution of the stress, see Art. 50, *d* 6. A section through the axis of a tension member should have a gross area in back of the pin equal to the net area of the transverse section.

Example

As an example of the method of computation, let it be required to design pin plates for a 5″ pin in a section carrying 396,000 lbs. compression, composed of one cover plate, 26″×½″; 2 web plates, 22″×½″; 2 top L's, 4″×4″×½″; and 2 bottom

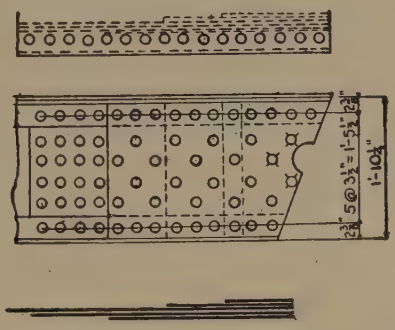


Fig. 117*b*.—Pin Plates as Computed.

L's, 6″×4″×½″. Allowable unit stress in shear, 7500; in bearing, 15,000 lbs. per sq.in.

We first find that the total length bearing required on one side is,

$$\frac{396{,}000}{2\times 5\times 15{,}000} = 2.64 \text{ ins.}$$

The ½″ web plate may be counted as part, also a 14×½″ outside web, leaving a balance of 1.64″. We will use three plates, each $\frac{9}{16}$″ thick. Assuming now ⅞″ rivets with a value

in single shear of 4500 lbs., we may construct the following table for half section, everything in kips and inches:

Section.	Area.	Stress.	Rivets.	Pin Plates.		
				Thickness	Stress.	Rivets.
1 Cov. Pl., $13''\times\frac{1}{2}''$....	6.50	49.5	11	$\frac{1}{2}''$	36.8	9
1 Web Pl., $22\times\frac{1}{2}$.....	11.00	83.7	19	$\frac{1}{2}$	36.8	9
1 Top L, $4\times4\times\frac{1}{2}$.....	3.75	28.6	7	$\frac{9}{16}$	41.5	10
1 Bott. L, $6\times4\times\frac{1}{2}$.....	4.75	36.2	8	$\frac{9}{16}$	41.5	10
				$\frac{9}{16}$	41.5	10
Totals.....	26.00	198.0	45	$2\frac{11}{16}$	198.1	48

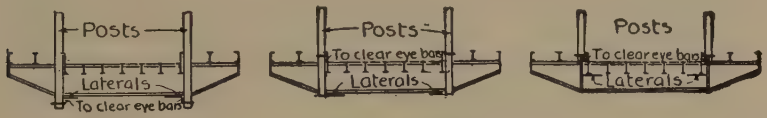
The design is worked out in Fig. 117*b*. The critical part of the problem is the upper row of horizontal rivets. These carry a total stress of $49.5+28.6=78.1$ kips, into top angle and cover plate. We have furnished 11 rivets in bearing on $\frac{1}{2}''$ and 3 in single shear with a total strength of 90.0 kips.

Prob. 117. For the top chord of Prob. 115, design a pin joint at end, the necessary splices of type (4), and the pin plates at the intermediate panel points for differences of stress as given.

Art. 118. Floors and Other Details

The floors for pin bridges are much like those for riveted. Panel lengths are likely to be somewhat longer, resulting in heavier work. Principal changes, however, are caused by the pin. Clearance must be provided for entering and driving this with short pilot and driving nut, Art. 46. A rough idea of the length in inches of the former may be given by the expression, $0.1D+2''$, where D is the diameter of the pin in inches. For the length of the latter, we can use $0.5D+4''$. On one side, we must have the grip of the pin plus pilot nut plus driving nut, and, in addition, a few feet for entering and driving. On the other side, we need room for unscrewing pilot nut. Clearances at struts may be obtained by erecting them after pin is driven.

Naturally, stringers will not be affected. Partly on account of the difficulty which would arise with sidewalk connections, it is customary to place floor beam either entirely above or below pin. Fig. 118*a* shows method of construction for the former. This is the best design, but may call for more depth than is



Highway Bridges.

FIG. 118*a*. FIG. 118*b*. FIG. 118*c*.

Floors for Highway Pin-connected Bridges.

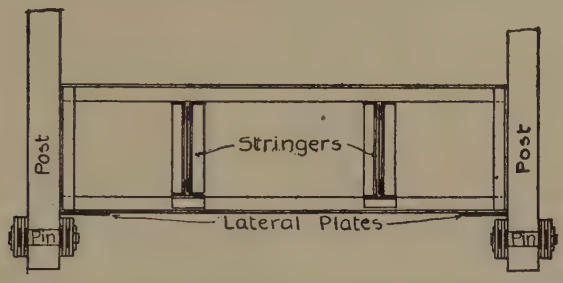


FIG. 118*d*.—Floor Beam for a Railroad Bridge.

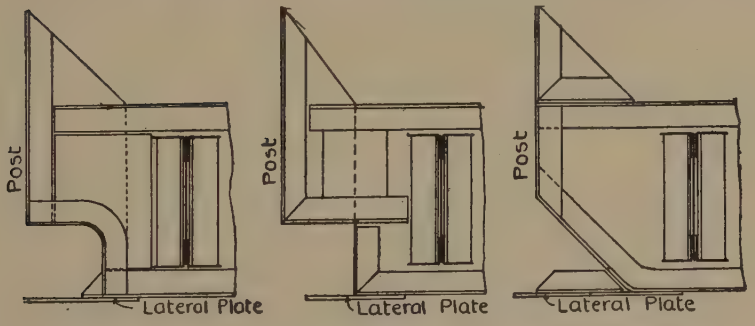


FIG. 118*e*. FIG. 118*f*. FIG. 118*g*.

Drop Floor Beams.

economical and joint for sidewalk bracket is unsatisfactory. Fig. 118*b* shows one arrangement with pin above. Here post is continuous. This has in addition to a difficult sidewalk

connection, same as for preceding, a location of the laterals which causes bending in the posts. The latter objection holds also for the suspended floor beam designed in Fig. 118*c*. See Fig. 59*f* for details. In this case, the hangers are so deficient in transverse strength that they render the laterals useless. Although it allows of a continuous floor beam, dispensing with the awkward sidewalk connection, the first objection is vital and this type has no place in good work.*

FIG. 118*h*.—Through Pratt Trusses for Interurban Electric Railroad Bridge at Des Moines, Iowa.

RAILROAD BRIDGES

The same types may be found in existing railroad bridges. However, the objections to ineffective lateral bracing is much more important† and form of Fig. 118*a* is the only one of the three which receives our approval. It is shown adapted for railroad bridges in Fig. 118*d*. To allow of a more efficient portal and to keep distance from base of rail to low steel down to a minimum, we often use the "drop" floor beam. Here

* This is so far conceded by modern design that the reviewer has termed this detail "obsolete."

† *Ibid.*

a notch is cut large enough for eyebars and withdrawal of the pilot nut. To make up for the web area lost and to give room for the necessary rivets connecting to posts, web is prolonged above or sometimes thickened by additional plates. A glance shows such floor beams to be expensive in material and shop work, yet it is very doubtful if they have anything like the full strength of the rectangular design of Fig. 118*d*. In other respects, however, it is excellent and widely used. The notch cut may be circular, Fig. 118*e*; square, Fig. 118*f*; or triangular, Fig. 118*g*. The latter is best, as there are no sudden changes of section and shop work is not too expensive.

Other Details

About the only difference from riveted trusses which we can note under this head is that care must be taken in lateral connections to leave room for eyebars and for pilot nuts.

Prob. 118. Design a drop floor beam for allowable stresses of Art. 50. Stringers are 24″ deep and 6′ 0″ c. to c. Tracks, 13′ 0″ c. to c. Trusses, 30′ 0″ c. to c. Maximum floor beam reaction, 86,000 lbs. per rail.

Art. 119. Camber

This has been defined and its need stated in Art. 90. We

Fig. 119*a*.

Fig. 119*b*.

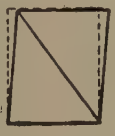


Fig. 119*c*.

Fig. 119*d*.

Methods of Cambering Trusses.

do not favor it for plate girders. For trusses whether pin or riveted, it should be employed, and the longer the span, the more imperative the need. There are not less than four methods. The first three keep upper and lower chord points of parallel chord trusses in the arcs of circles, those for (1) and (2) being concentric.

(1) Fig. 119*a*. Lengthen upper chord and shorten lower chord the same amount, usually $\frac{1}{16}''$ for 10 feet, leaving web members unchanged.

(2) Fig. 119*b*. Lengthen upper chord by about $\frac{1}{8}''$ for every 10 feet, leaving posts and lower chords unchanged. Alteration in length of diagonals must be computed.

(3) Fig. 119*c*. Change diagonals only.

(4) Fig. 119*d*. Compute deformation of each main member under load which causes maximum deflection of entire truss and make fabricated length such that under said load it becomes equal to theoretical length.

Prob. 119. A deck Pratt truss of type 2, Fig. 103*c* has 8 panels at 20 feet and a depth of 25 feet. Give fabricated length for all members for a camber of 2″ by each of the first three methods.

Art. 120. Computations

Loads and specifications are similar to those already given.

The floor is first computed in the usual manner. Next the trusses are designed beginning with the posts. The principal point is to secure economy of material by keeping radii of gyration the same in both directions. Be sure that sufficient room is provided for floor beam connections and diaframs. It will be found difficult to secure good rivets with channels less than 8″ in depth for highways and 10″ for railroads. Out to out of webs of posts should be made constant to avoid varying floor beams. Gage to gage of channels should also be kept the same so that there is but one kind of lattice bar.

In designing the eyebars, we must provide the necessary rectangular area, with a ratio of depth to breadth lying between 4 and 8. For counters, it is common to use single bars or perhaps loop-ended square rods. Main members, however, must be in pairs, two, four, six, and so forth. The advantages of using more than two are the smaller pins and clearances required. The disadvantages are the increased stresses due to own weight and the greater danger of an *unequal distribution of the stress*. Labor and worry may be avoided by using as few sizes as

possible. To guide the beginner, we may try as a rough rule of thumb, depths of $5''+\frac{\text{span}}{1200}$, using rather more for heavy bridges and rather less for light. This tentative depth may be discarded for a more satisfactory one after the preliminary design.

We next determine sections for stiff tension members. Transverse distances must, of course, be kept constant for each separate piece, this distance being determined by a consideration of all members affected.

The top chord should be wide enough in the clear to admit connections and have room above neutral axis for largest eyebar head. It is our problem to design a section with these governing dimensions which will have the utmost theoretic and practical economy.

In order to get size of eyebar head as above, we must know size of pin. The best that can be done is to assume its diameter as something more than three-fourths of the depth of the deepest bar. A good size to try is the largest pin which may be used in smallest standard sized eyebar head. If size as determined below does not agree, we must revise and recompute until assumptions and results coincide.

Computation of Pin

A pin is treated like a round beam acted upon by non-concurrent, non-parallel forces perpendicular to its axis. We proceed as follows to obtain its diameter at any joint:

(1) Ascertain if there is sufficient bearing area on pin of assumed diameter to carry maximum stress. If not, said area must be increased by the addition of pin plates, Art. 117. This need not be done for eyebars since the rule for the minimum diameter of pin takes care of it.

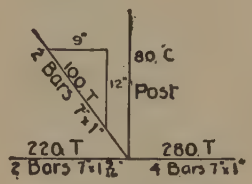


Fig. 120a.—Problem.

(2) Locate each load in a direction parallel to the pin. See Art. 117 for clearances to be allowed.

(3) Find stresses in each connecting member due to the load which will try the pin most severely. This may sometimes be seen at inspection; otherwise, we try various combinations; the usual ones are that giving maximum moment at joint and that giving maximum shear in an adjacent panel. Consider each force applied at the center of its bearing area.

(4) Resolve each force into horizontal and vertical components.

(5) Draw diagram of horizontal and vertical shears.

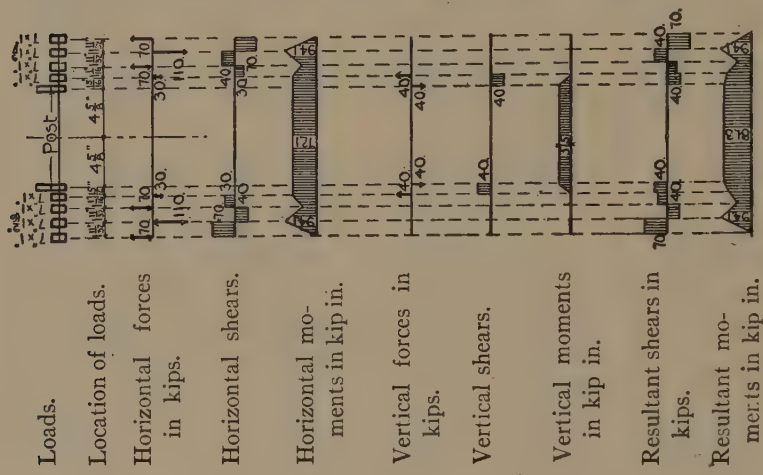


FIG. 120*b*.

(6) Draw diagram of resultant shears.

(7) Draw diagram of horizontal and vertical moments.

(8) Draw diagram of resultant moments.

(9) Select pin which will stand maximum resultant shear and moment.

(10) Any other possible combinations must be similarly tried.

(11) If this does not agree with assumed diameter of pin, a revision may be necessary.

It is customary to employ not more than two sizes of pins in a bridge. Using few depths of bars and diameters of pins result

in numerous economies which more than offset extra material. Hence, in practice, all pins are not figured, but only those which may have maximum shear or moment.

Example

Let it be required to compute problem seen in Fig. 120*a*. Allowable stresses in shear, 7,500 lbs. per sq.in.; flexural, 15,000.

The diameter required in shear is $3\frac{1}{2}$; in bending, 4. But the bearing of the eyebar will require $\frac{3}{4}\times 7 = 5\frac{1}{4}''$ and this is the required size.

Prob. 120. Solve the same problem when counter, a single bar, $4''\times 1''$, is in action with a stress of 25,000 lbs. at same angle with the vertical but in the opposite direction. The stress in right-hand bottom chord member is then 120,000 lbs.

Art. 121. Design of a Pin-connected Through Highway Span

Let it be required to design a span of 200 feet with roadway at least 22 feet wide in the clear, and no sidewalks. It is to be computed for a heavy traffic in light vehicles.

We will use an asphalt floor; 3″ will weigh about 25 lbs. per sq.ft.; 5″ average concrete, 60 lbs.; $\frac{5}{16}''$ steel buckle plates, 13 lbs. Allowing 12 lbs. for stringers, we get 110 lbs. per sq.ft. dead. We will use this throughout, adding 140 lbs. for live. This practically amounts to decreasing the loads on the floor beam by the weight of the floor beam itself, and those on the truss by weight of floor beam and truss. Since the solid floor in the plane of the top of the floor beam acts as a lateral system, we will place pins just above the floor. The upper laterals and portals will be designed for 40 lbs. per sq.ft. wind pressure. Use allowable unit stresses for soft steel, as follows:

Tension or flexure..... 12,000

Compression..... $12{,}000 - 60\frac{l}{\rho}$

Shear on shop rivets and net section of webs	9,000
Shear on field rivets	6,750
Bearing on rivets	Twice above values
Bearing on masonry	400

For Medium Steel

Shear on pins	10,000
Bearing on pins	20,000
Flexure on pins	20,000
Bearing on rollers	400 × diameter in inches

General Dimensions

Make the transverse spacing of the trusses 25′ 0″. Using 7 for n in (7a) of Art. 102, we find economical panel length to be nearly 16 feet. We shall figure the 7 I-beam stringers to each carry 4 feet in width or 1000 lbs. per lin.ft. One I, 12″ @ 31.5# just suffices for 12 panels at 16′ 8″ and we will consider this as fixed. Using (3) of Art. 102, we derive that h equals 24′ 2″. A height of 18′ 0″ at hip will give 5 feet for portal and almost 14 feet for clearance. We will use 30 feet depth at center, giving the truss of the form shown in Fig. 121a.

Floor Beam

The load on the floor beam is 4150 lbs. per lin.ft. Use 36″ web. Maximum shear 52,000 lbs., use $\frac{3}{8}$″ * web. By the intermediate method, we find necessary two L's, 4″ × 4″ × $\frac{5}{8}$″, top and bottom. Spacing of $\frac{3}{4}$″ rivets, approximate method; end, 3.33″; 6 feet out, 6.06″.

The live panel load is 29 kips and the dead is 23. Of the latter, we shall consider 5 as applied to the top chord and 18 on bottom. We may now tabulate, keeping all values in kips † and inches.

* $\frac{5}{16}$″ would do, but $\frac{3}{8}$″ increases rivet value and helps carry the moment.

† One kip = 1000 lbs.

Posts

Member.	Stresses.			Use 2 [’s.	Area.	ρ.	Allowable Stresses.	
	Dead.	Live.	Total.				Unit.	Total.
U_1L_1	18.0T	29.0T	47.0T	10″ @ 20	11.76	.....	10.0*	118
U_2L_2	43.5C	69.1C	112.6C	10 @ 30	17.64	3.42	7.4	131
U_3L_3	14.4C	46.8C	61.2C	10 @ 20	11.76	3.66	6.9	81
U_4L_4	9.6T	31.5C	21.9C 47.0T	10 @ 20	11.76	3.66	6.1 10.0*	72 118
U_5L_5	16.5C	50.7C	67.2C	10 @ 20	11.76	3.66	6.1	72
U_6L_6	5.0C	36.2C	41.2C	10 @ 20	11.76	3.66	6.1	72

Channel flanges are turned out.

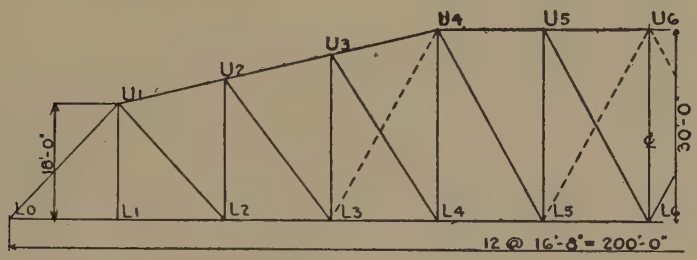


FIG. 121*a*—Truss of Problem.

Bottom Chords

Member.	Stresses.			Area. Required.	Use.	Area Furnished.
	Dead.	Live.	Total.			
L_0L_1	117T	148T	265T	22.1	2, 7″×1 5/8″	22.75
L_1L_2	117T	148T	265T	22.1	2, 7 ×1 5/8	22.75
L_2L_3	174T	220T	394T	32.8	4, 7 ×1 3/16	33.25
L_3L_4	199T	251T	450T	37.5	4, 7 ×1 3/8	38.50
L_4L_5	204T	258T	462T	38.5	4, 7 ×1 3/8	38.50
L_5L_6	224T	283T	507T	42.2	4, 7 ×1 1/2	42.00

* Reduced to allow for holes.

Diagonals and Counters

Member.	Stresses.			Area Required.	Use.	Area Furnished.
	Dead.	Live.	Total.			
U_1L_2	84.1T	114.8T	198.9T	16.6	2, 7×1 3/16	16.6
U_2L_3	40.6T	73.7T	114.3T	9.5	2, 5×1	10.0
U_3L_4	10.0T	48.2T	58.2T	4.8	2, 4×5/8	5.0
U_4L_5	39.4T	77.6T	117.0T	10.0	2, 5×1	10.0
U_5L_6	13.1T	58.2T	71.3T	5.9	2, 4×3/4	6.0
U_4L_3	.....	.....	28.2T	2.4	1, 4×5/8	2.5
U_6L_5	13.1C	41.5T	28.4T	2.4	1, 4×5/8	2.5

The largest eyebar connecting to top chord is 7″ in depth. Unless pin exceeds 7″, which is improbable, the diameter of the head in the American Bridge Co. Standards is $16\frac{1}{2}$″. The center of gravity of the section should therefore be below cover plate more than $8\frac{1}{4}$″. The channel posts will be made 7″ out to out of webs, which makes least radius of gyration in the other direction in all cases. Greatest width will be required at U_1. Allowing for two $\frac{3}{8}$″ hinge plates, two $1\frac{3}{16}$″ eyebars, and clearances, distance inside of plates should be not less than 12″. We will use section seen in Fig. 121*b*.

Fig. 121*b*.—Section of Upper Chord.

Top Chord

Member.	Stresses.			Use.	Area.	ρ.	Allowable Stresses.	
	Dead.	Live.	Total.				Unit.	Total.
L_0U_1	173C	218C	391C	1 Pl. 21×7/16 2 Pls. 20×1/2 2 L's 3½×3½×7/16 2 L's 5×3½×7/16	42.0	6.80	9.40	395C
U_1U_2	180C	227C	407C				10.18	427C
U_2U_3	205C	259C	464C	Above section plus 2 Pls. 11×3/8	50.25	6.80	10.18	511C
U_3U_4	211C	266C	477C				10.18	511C
U_4U_5	224C	283C	507C				10.23	515C
U_5U_6	230C	290C	520C				10.23	515C

Wind Bracing

The solid floor is ample for plane of bottom chord. For the top chord,* the maximum stress in the upper intermediate struts is 27.0 kips compression, and unsupported length is 25.0 feet. Four L's, $4'' \times 3'' \times \frac{5}{16}''$, of a depth equal to the top chord and with shorter legs latticed together $\frac{5}{8}''$ apart, is the smallest section which answers. This is not quite enough, so we will increase thickness to $\frac{3}{8}''$ and use this throughout.

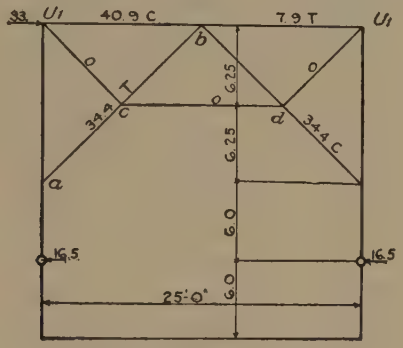


Fig. 121c.—Loads and Stresses in Portal.

Diagonals

Here we will use one L, $4'' \times 3''$ with the longer leg vertical. We may now prepare the following table, keeping everything in kips and inches. The $\frac{3}{4}''$ field rivets will have a value of 3000 lbs.

Member.	Stress.	Use Thickness, Ins.	Areas.		Capacity.	Rivets.
			Gross.	Net.		
U_1U_2	32.7T	$\frac{1}{2}$	3.25	2.81	33.7T	11
U_2U_3	25.5T	$\frac{3}{8}$	2.48	2.16	25.9T	9
U_3U_4	18.2T	$\frac{5}{16}$	2.09	1.82	21.8T	6
U_4U_5	10.9T	$\frac{5}{16}$	2.09	1.82	21.8T	4
U_5U_6	3.6T	$\frac{5}{16}$	2.09	1.82	21.8T	3

* Upper diagonals figured to take tension only, panel load 6.0 kips.

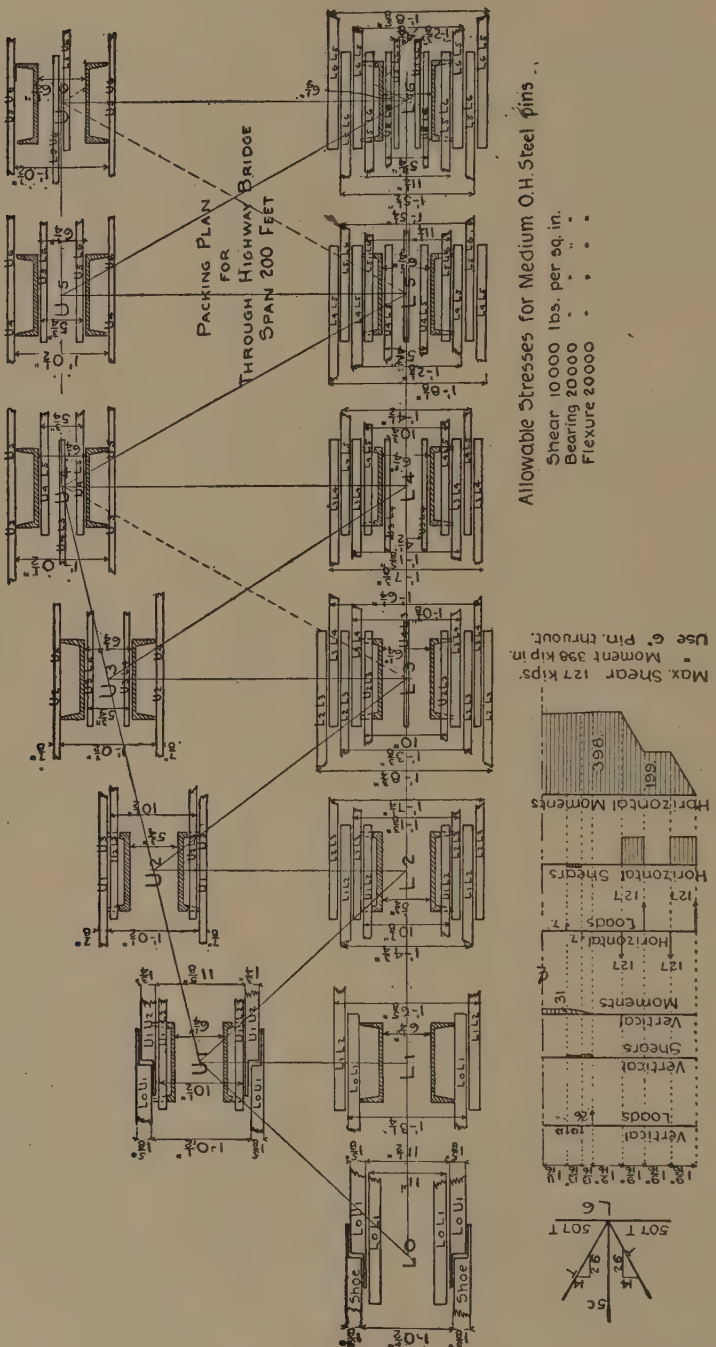


Fig. 121*d*.—Packing Plan.

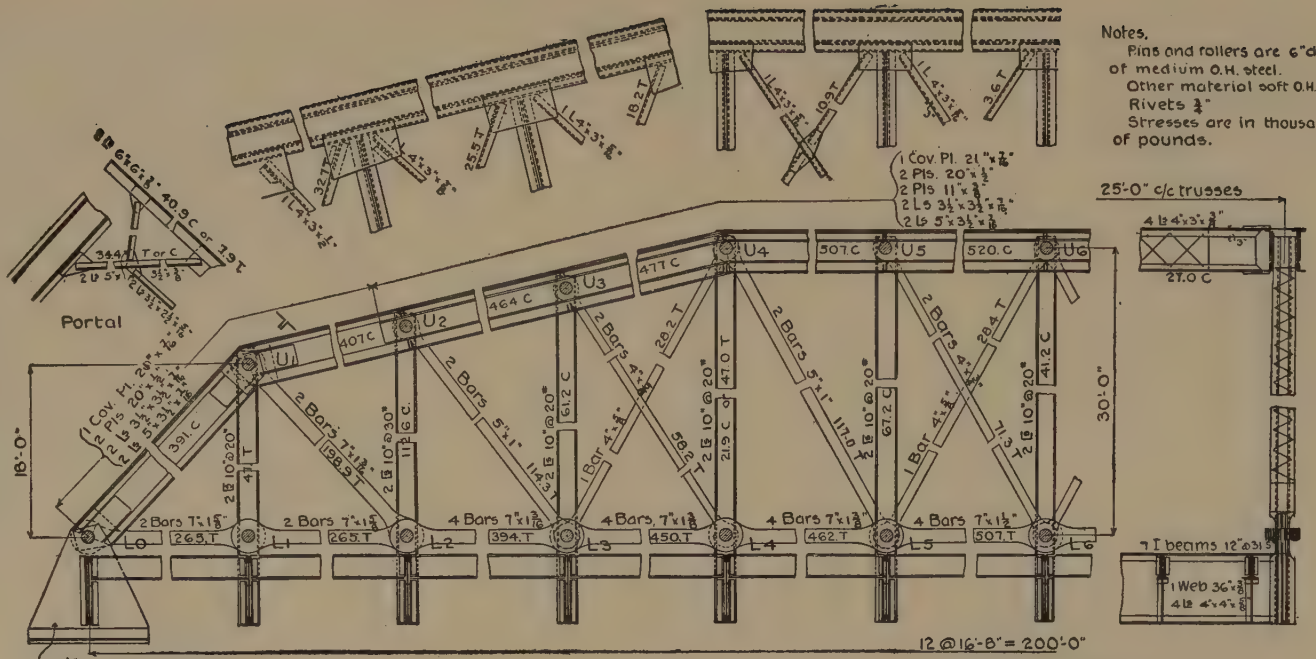


Fig. 121a.—General Drawing.

Portal

U_1c and cd. Maximum length, 12.5 feet. Minimum radius of gyration, 1″. Use two L's, $3\frac{1}{2}\times 2\frac{1}{2}\times\frac{5}{16}''$, longer legs together.

ab. Maximum stress, 34.4 T or C. Unsupported lengths are 8.9 feet one way and 17.7 feet the other. Use two L's, $5''\times 3\frac{1}{2}''\times\frac{3}{8}''$, $\frac{3}{8}''$ apart, shorter legs together, gross area 6.10 sq.in., and radii of gyration equal to 1.03 and 2.40″. Allowable stress, 65.5 T and 35.2 C.

U_1U_1 maximum stresses, 40.9 C or 7.9 T. Unsupported lengths, 12.5 and 25.0 feet. Use two L's, $6''\times 6''\times\frac{3}{8}''$, $\frac{3}{8}''$ apart, gross area 8.72 sq.in., minimum slenderness ratio, 114, allowable compression, 45.0. O.K.

Bending in post, $16.5\times 72 = 1190$ kip in. $\frac{I}{c}$ for end post in this direction is 189. Unit stress in extreme fibre, 6300. While this in conjunction with the dead will produce stresses somewhat higher than those allowed by specifications, we shall permit it to stand. The bracing effect of the portal in the weaker plane of the post, and the rare occurrence of a wind producing a pressure of 40 lbs. warrants the use of the unchanged section.

Shoes

Maximum reaction 309.5

Bearing area required $\frac{309.5}{.400} = 776$ sq.in.

Lineal inches of 6″ rollers $= \frac{309.5}{6\times .400} = 129$

Packing Plan. Fig. 121*d*

This shows the way in which the different members connect and should be carefully studied. Easier clearances, and some economy of material would result if top chord were made 1″ wider. It should not be forgotten, however, that compact joints make a stiffer, better structure.

Computation of Pins

At the same time that the packing plan is made the pins must be figured, the two being interdependent, as explained in Art. 120. Although several pins were figured, we have given in Fig. 121*d* computation for the one with the highest stresses only.

General Drawing

The general drawing for this truss is given in Fig. 121*e*.

Art. 122. Design of a Single-track Pin-connected Railroad Bridge

Let it be required to design a through bridge of 300 feet span for a single-track railroad. Live load for floor, Cooper's E 60; for trusses, 4000 lbs. per lin.ft. per rail. Wind load, 30 lbs. per sq.ft., dead on top chord and live on the bottom. Train to be considered as 10 feet high. Allowable stresses for all loads in lbs. per sq.in.

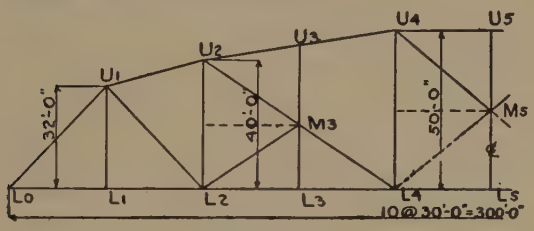


Fig. 122*a*.
Truss of Problem.

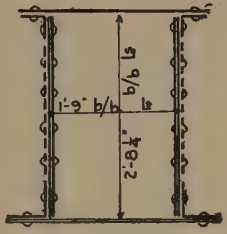


Fig. 122*b*.
Section of Top Chord.

	Soft Steel.	Medium Steel.
Tension or flexure		12,000
Compression		$12{,}000 - 50\frac{l}{\rho}$
Shear on rivets, shop	7,500	
Shear on rivets, field	6,000	
Bearing on rivets, shop	15,000	
Bearing on rivets, field	12,000	
Shear on pins or gross section webs		9,000
Bearing on pins		18,000
Flexure on pins		20,000
Bearing on masonry		300
Rollers per lin.in		300 times diameter in inches.

We will take center to center of trusses as 20 feet. A closer spacing would clear, but this is about the least for the proper stability. The economic panel length is $\frac{L}{6}\sqrt[3]{\frac{8000}{2 \cdot 12{,}000}} = \frac{L}{8.64}$. However, the fact that we are using heavier loads for the floor beam and the partial use of the secondary system will decrease this distance and we will try 10 panels @ 30 feet. The economical depth for a parallel chord bridge will be $0.42\sqrt{30 \cdot 300} = 40$ feet. At end we must make it not less than 32 feet (4 feet stringer, 21 feet clearance, and 7 feet portal). To get the mean depth equal to the economical, we will use 50 feet at the middle, giving the form seen in Fig. 122*a*.

Stringers

Assumed dead load per lin.ft., 200 lbs. for one-half of track and 300 lbs. for one stringer. Maximum end shear 102,000 lbs. Maximum moment, 8,070,000 in.lbs. Economical depth, 45″; use a 48″ web. $\frac{3}{8}$″ will furnish sufficient area for shear. Its equivalent in moment is 2.25 sq.in. Use two L's, $6'' \times 6'' \times \frac{11}{16}''$, top and bottom, with $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ stiffeners.

Floor Beams

Stringers spaced 7′ 0″ apart, c. to c.

Floor beam assumed to weigh 8000 lbs.

Allowing 500 lbs. per lin.ft. for the weight of one stringer, and one-half of the track, we have maximum floor beam reactions of 145,000 lbs., maximum shear of 149,000 lbs., and a maximum moment of 11,550,000 in.lbs. Using intermediate method, we require one web plate, $60'' \times \frac{1}{2}''$; four L's, $6'' \times 4'' \times \frac{1}{2}''$, and four Pls. $10'' \times \frac{1}{2}''$. The 6″ leg is to be placed vertically and is needed to carry the rivets for horizontal shear.

Panel Loads

Dead for floor equals 19, say 20 kips.

Dead for truss, Art. 102 48 kips. Part of (9), $w = 10{,}000$

Total.....68 kips, 24 top, 44 bottom.

Live on bottom chord.....120 kips

Wind on bottom chord..... 20 kips

Wind on top chord..... 10 kips

We may now tabulate stresses, keeping everything in kips and inches. For the secondary verticals and diagonals, we shall use maximum floor beam reaction for locomotive loading instead of that due to uniform live load. The wind on the top chord will be assumed to be carried down end posts by the portal. We will make posts with flanges turned in and a back to back of 12.5″, which will give minimum radius of gyration in other direction.

Posts

Member.	Total Stresses.				Use 2 [′s.	Unsupported Length.	ρ.	Area.	Allowable.	
	Dead.	Live.	Wind.	Sum.					Unit.	Total.
U_1L_1	44T	130T	0	174T	15″ @ 33 #	.....	....	19.80	10.0*	198T
U_2L_2	51C	103T	14.4T	66T	15″ @ 40 #	240	5.43	23.52	10.0*	235T
		151C	14.4C	216C					9.79	230C
U_4L_4	23T	120T	10.3T	153T	15″ @ 33 #	300	5.62	19.80	10.0*	198T
		108C	10.3C	95C					9.34	185C
U_3M_3	24C	0	0.0	24C	10″ @ 20 #	300	3.66	11.76	7.90	93C
U_5M_5	24C	0	0.0	24C						
M_3L_3	44T	130T	0.0	174T	15″ @ 33 #	.....	....	19.80	10.0*	198T
M_5L_5	44T	130T	0.0	174T						

* Reduced to allow for rivet holes.

Bottom Chord. 12.0 kips per square inch allowable

Member.	Total Stresses.				Area Required.	Use.	Area Fur.
	Dead.	Live.	Wind.	Sum.			
L_0L_1	287T	507T	158T	952T	79.3	2 Eyebars, 12″×2″ 2 Pls., 24×⅝″ 4 L's, 4″×4″×⅝″	Gross 96.4
L_1L_2	287T	507T	278T	1072T	89.3	2 Eyebars, 12″×2″ 2 Pls., 24″×⅝″ 4 L's, 4″×4″×⅝″	Gross 96.4
L_2L_3	459T	811T	381T	1651T	137.6	6 Eyebars, 12″ @ 1 15/16	139.5
L_3L_4	459T	811T	441T	1711T	142.6	6 Eyebars, 12″×2″	144
L_4L_5	490T	865T	486T	1841T	153.4	6 Eyebars, 12″×2⅛″	153

At pin holes of built portion of L_0L_1, and L_1L_2, extra material must be provided, see Art. 50, *d* (6). Care should also be taken in detailing that net area is not less than 79.3 and 89.3 respectively.

Diagonals

Member.	Total Stresses. Dead.	Live.	Wind.	Sum.	Use.	Area.	ρ.	Allowable. Unit.	Total.
U_1L_2	177T	356T	19T	552T	2 Eyebars $12'' \times 1\frac{15}{16}''$	46.5	....	12.0	558T
U_2M_3	99T	277T	18T	394T	2 Pls. $24'' \times \frac{9}{16}''$	40.0	8.50	10.0*	400T
		104C	18C	23C	4 L's $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$			9.5	380C
M_3L_4	36T	272T	18T	326T	2 Pls. $24'' \times \frac{9}{16}''$	40.0	8.50	10.0*	400T
		207C	18C	189C	4 L's $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$			9.5	380C
U_4M_5	53T	281T	0	336T	2 Eyebars $8'' \times 1\frac{3}{4}''$	28.0	....	12.0	336T
M_5L_4	0	187T	0	187T	2 Eyebars $8'' \times 1\frac{1}{8}''$	18.0	....	12.0	216T
M_3L_2	61C	117C	0	178C	2 [s 15" @ 40#	23.5	5.43	8.0	188C

* Reduced to allow for rivet holes.

Upper Chord

Adding to the $12\frac{1}{2}''$ for posts, $4''$ for eyebars, and $2''$ for hinge plates and clearances, we require $18\frac{1}{2}''$ clear inside. Leave about $14''$ for eyebar head.

Member.	Total Stresses. Dead.	Live.	Wind.	Sum.	Use.	Area.	ρ.	Stress. Unit.	Total.
L_0U_1	420C	742C	82C	1244C	1 Top Pl., $30 \times \frac{11}{16}''$ 4 Web Pls., $32 \times \frac{5}{16}''$ 2 Web Pls., $22 \times \frac{5}{8}''$ 2 Web Pls., $30 \times \frac{3}{8}''$ 2 Top L's $4 \times 4 \times \frac{5}{8}''$ 2 Bott. L's. $6 \times 6 \times \frac{5}{8}''$	166.1	10.4	7.6*	1262C
U_1U_2	422C	745C	10C	1177C	1 Top Pl., $30 \times \frac{11}{16}''$ 4 Web Pls., $32 \times \frac{9}{16}''$ 2 Top L's $4 \times 4 \times \frac{5}{8}''$ 2 Bott. L's, $6 \times 6 \times \frac{5}{8}''$	116.1	10.2	10.2	1184C
U_2U_3	497C	877C	20C	1394C	U_1U_2 +2 Pls., $22 \times \frac{9}{16}''$	140.8	10.3	10.2	1436C
U_3U_4	497C	877C	52C	1426C	U_1U_2 +2 Pls., $22 \times \frac{9}{16}''$				
U_4U_5	531C	937C	65C	1533C	U_1U_2 +2 Pls., $22 \times \frac{3}{4}''$	149.0	10.3	10.3	1535C

* The portal is taken as leaving 26.0 feet unsupported, giving an allowable stress of 10.5 kips per sq.in. Moment due to the transverse wind load, Fig. 122*c*, assuming post as fixed at both ends, is $22.5 \times 13 \times 12 = 3510$ kip in. $\frac{I}{c}$ for section is 1210. Resulting flexural stress = 2.9 kips per sq.in. Allowable in pure compression, 7.6 as given above.

Laterals

The maximum stress in the upper diagonals is 32.3 kips T or C. Unsupported length, 18.4 feet. Minimum allowable radius of gyration is 221/125 equals 1.77 in.; four L's, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, $\frac{3}{4}''$ apart in pairs latticed together, have a gross area of 9.92 sq.in., a capacity of 57.0 kips in compression and more in tension. Although there is considerable excess, it is the minimum allowable size. For the struts, the stress is only 5 kips C. The length is 20 feet and required radius of gyration,

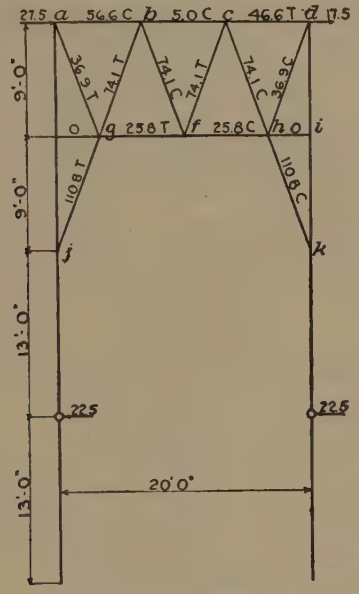


Fig. 122c.—Portal.

240/125 equals 1.92 ins. Use four L's, $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, with the short legs latticed together.

The stresses in the lower diagonals, beginning at panel nearest abutment, are:

81, 66, 50, 38, and 27 kips either T or C.

Use two L's $5'' \times 3\frac{1}{2}''$ with 5″ legs in contact. The average radius of gyration for this combination is about 1.37″, unsupported length, 150″, and allowable unit stress of 6600 compres-

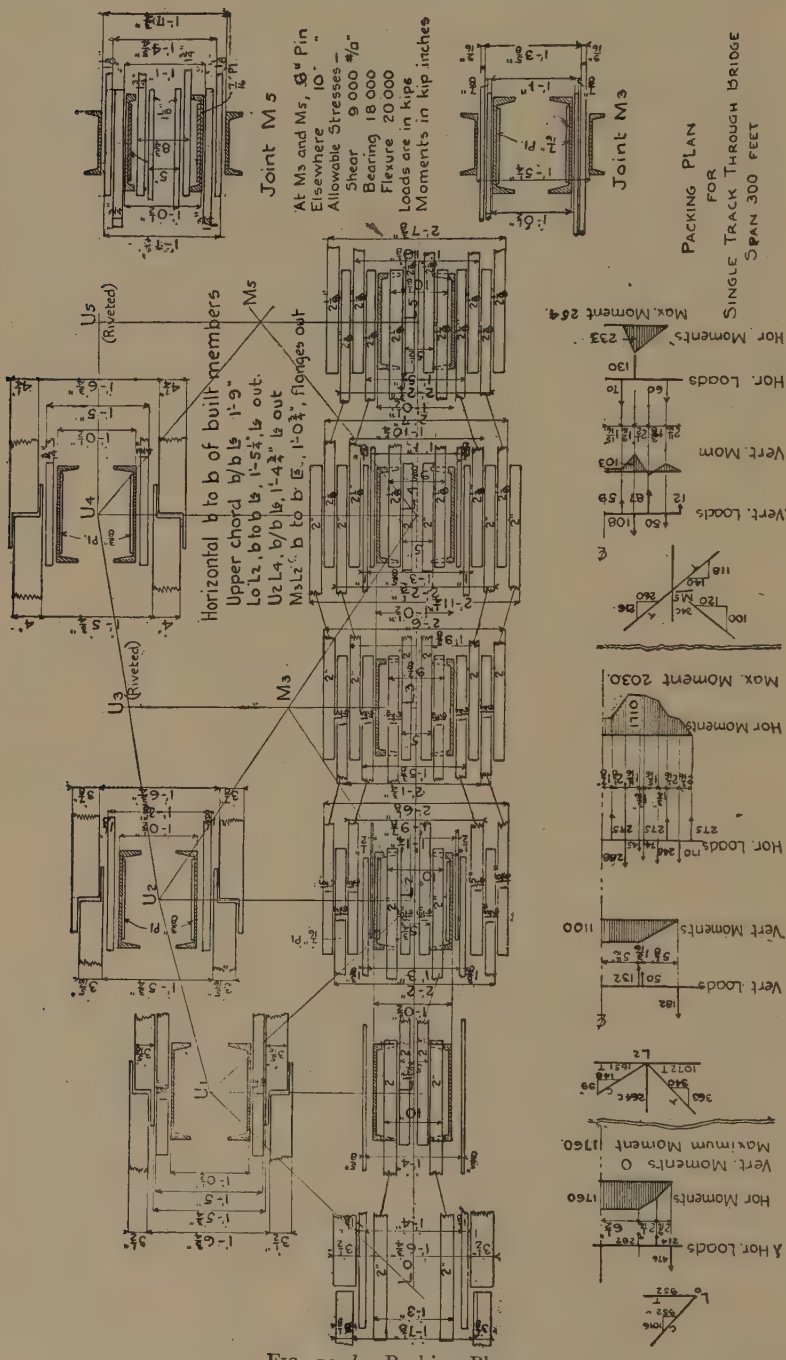
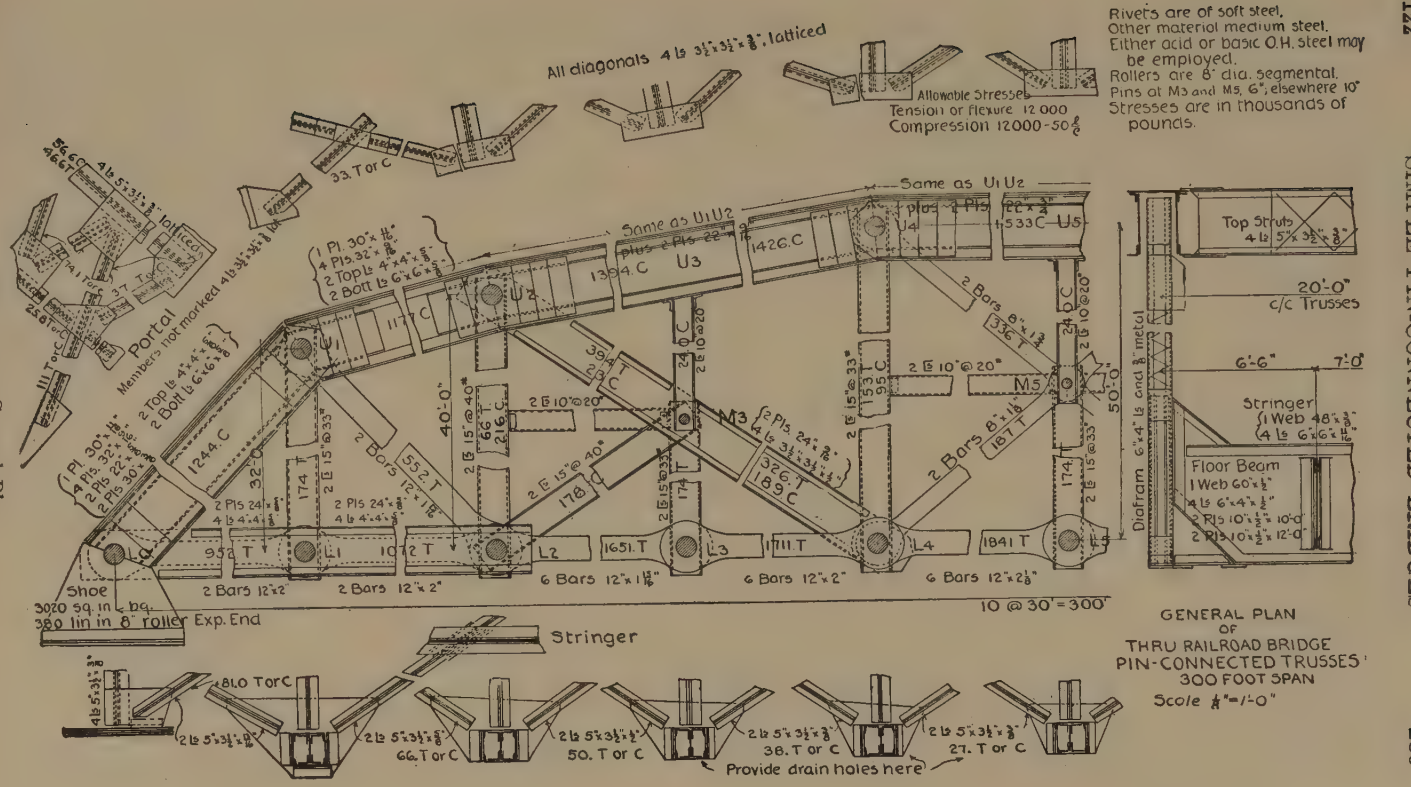


FIG. 122*d*.—Packing Plan.



GENERAL PLAN
OF
THRU RAILROAD BRIDGE
PIN-CONNECTED TRUSSES
300 FOOT SPAN
Scale ⅛"=1'-0"

FIG. 122c.—General Plan.

sion. The latter will require larger sizes than tension and we may now compute area required as follows:

12.3, 10.0, 7.6, 5.8, and 4.1 sq.in.

and we will use for the thickness:

$\frac{13}{16}$, $\frac{5}{8}$, $\frac{1}{2}$, $\frac{3}{8}$, $\frac{3}{8}$ ins. respectively.

The end struts, like the upper struts, are determined by their radius of gyration and four L's $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ forming a latticed I section, will be employed.

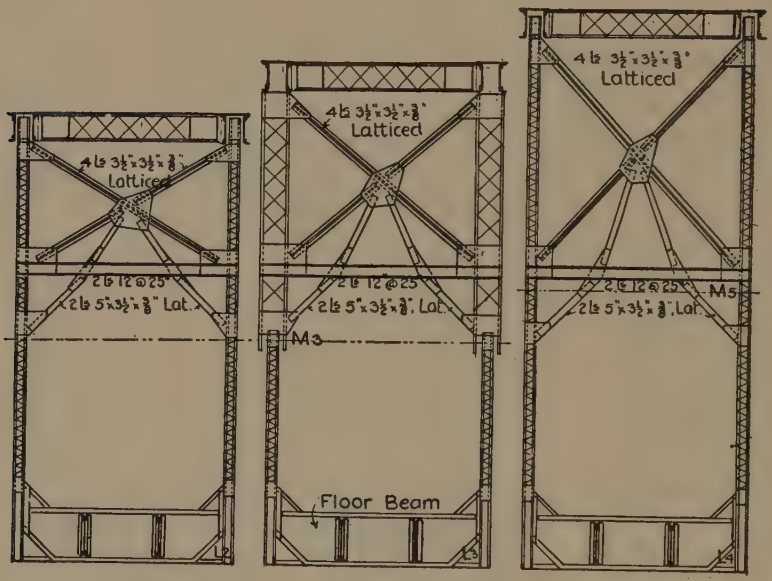


FIG. 122*f*.—Bracing at U_2L_2 at U_3L_3 at U_5L_5.

PORTAL

We will use here the four-angle section, of the same depth as the chord, with $\frac{1}{2}''$ double latticing at 45°. Four L's, $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ have a minimum radius of gyration of 2.65 ins. and a capacity for *ad* of 128.0 kips in compression, and 129.0 in tension. For *gh*, 4 L's, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ will carry 96.5 C or 98.6T.

For web members, the same section will stand 88 C and 98.6 T. This will be employed for *ag*, *bf*, *fc*, and *do*, as these are wind stresses. For *bj* and *ck* we find it necessary to use four L's $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ with a capacity in compression of 120.0 and 129.0 in tension.

Computations for Pin

Let us first examine truss and find joints where pin will be most heavily stressed. Such spots exist at L_0 and L_2. Consider the pins here to be $\frac{3}{4}$ of 12 equals 9″ in diameter. We now design pin plates on this basis and then compute pins to be 10″. Redesigning pin plates and solving afresh, we still find 10″ required. This size has a capacity in shear of over 700 kips. An examination of the packing plan, Fig. 122*d*, shows us that there will be no such shear, therefore it is not considered in the diagrams on the same figure. At intermediate joints, a smaller size may be used. A 6″ pin will be required at M_5 because it connects to 8″ bars and it is ample. Same diameter will be employed at M_3.

In obtaining size of pin plates in posts, note that we must use, not the stress in the body, but the maximum in the part between floor beam and the pin. One way is to use the maximum algebraic sum of all vertical components of diagonals at this point. The proper position of the live load may be obtained by trying a single load at any joint which may be in doubt.

CHAPTER X

STEEL MILL BUILDINGS

Art. 123. General

MILL buildings may be of wood, steel, or reinforced concrete. Brick, stone, or concrete masonry is often used for parts of these types. For the design of wooden buildings, see Chapter III; for reinforced concrete shops, the reader is referred to works on the subject.

Costs vary much with time and location. We will usually find wood cheaper than steel. Reinforced concrete will probably cost less than protected steel and more than unprotected. Concrete, it must be remembered, is difficult to alter and is entirely unsuited for temporary buildings or those which may need extensive alterations from time to time. On the other hand, it is durable and well resists fire and the weather.

Directing now our attention to steel structures, we meet the following types:

(1) *Steel Frame.* Fig. 123*b*.

Here the roof trusses, columns, beams, and bracing are made of steel.

(2) *Steel Frame with Masonry Filled Walls.* Bracing and siding in plane of columns are replaced by a wall of concrete, brick, stone, or tile. Masonry is much more expensive than bracing and siding, but it lasts longer, protects the steel against rust and fire, and better excludes the weather. Stone masonry is costly and less fire resisting, hence seldom used. Tile is light and cheap but not very strong. Brick and concrete are fire resisting, weather proof, and durable. A choice of these two will be determined by the cost, which varies with the locality.

(*2a*) A modification of the preceding is the type in which the walls are plastered on specially formed expanded metal such as Hy-Rib, Self-centering, etc. This method makes a wall which, while good, is comparatively cheap.

(3) *Solid Masonry Walls.* These, whether of concrete, brick, or stone, carry the loads from the roof trusses without the assistance of metal. They are fireproof and weather resisting, but will be more expensive than either of the other types except for low walls.

FIG. 123*a*.—Shop of Midvale Steel Co. McClintic-Marshall Construction Co., Rankin, Pa.

Buildings are usually rectangular in plan, occasionally with one or more rectangular additions. The size of the lot is sometimes a limiting factor. The important point, however, is to design the building to suit its prospective contents and processes. Hence the outside dimensions, heights in the clear, number and capacity of cranes are determined by an engineer whose field is outside the scope of this work.

Site should be chosen where labor is cheap and plentiful and land for future extensions and alterations will be available without excessive cost, and where cars can be run directly into the works that material may be unloaded and output shipped with as little handling as possible. Connection with a competing road or waterway is also desirable. Location should be such that means of intercommunication between the

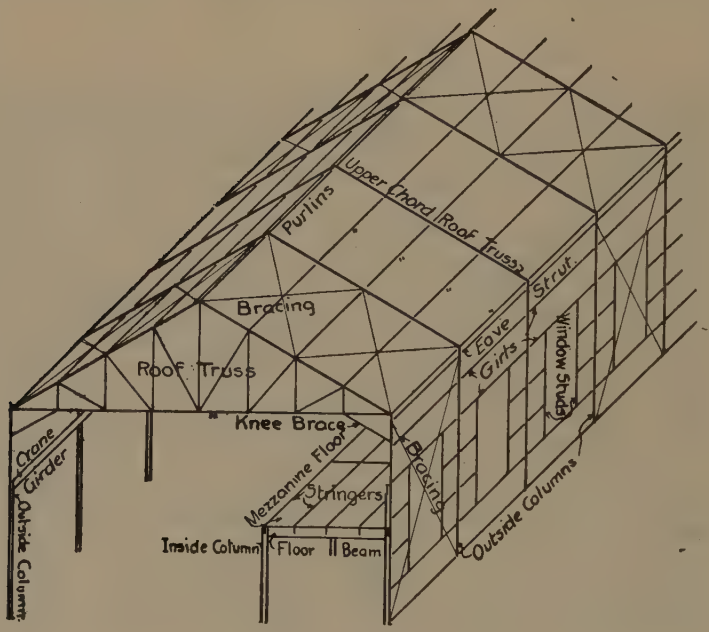


FIG. 123*b*.—Steel Frame Mill Building.

different buildings of the same plant, such as elevators, conveyors, cranes, and yard railways, may be cheaply installed.

A single story building is preferable since (1) it is a lesser fire risk, (2) is safer to employees, (3) material is more easily handled, (4) machinery causes less vibration, and (5) light is much better. Its disadvantages are, (1) more expensive per square foot of floor area, (2) the added cost of land, (3) increased amount of grading, (4) loss of power through transmission.

With these considerations in mind, the typical mill building has been evolved. In the steel frame construction, the outside covering, Art. 124, very often of corrugated steel on steel beams, is carried at the top by steel trusses, Art. 125, placed 10 to 30 feet apart. These trusses are in turn carried by columns, Art. 126, which also sustain the weight of the siding. Trusses and columns must be thoroughly braced together, Art. 127. Provision will often be required for bridge and jib cranes, Art. 129.

Transversely, allowance for expansion is made in the case of roof trusses only in buildings of type (3). Even here it is often omitted for spans less than 75 feet. $\frac{1}{8}''$ in 10 feet is the usual allowance. Longitudinally, practice varies a great deal from omitting all such provision to inserting an expansion joint in every other panel. Perhaps every hundred feet is the best practice. It is usually made by slotting the holes at one end of all longitudinal members in a *bay* as the length between trusses is called.

This distance is determined by economical considerations. The approximate weight of one roof truss may be obtained from the empirical formula,

$$R=(wa/S)^{0\cdot 7}(6L^2+120L), \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where R = weight of one truss in pounds;
w = equivalent horizontal uniform load in lbs. per sq.ft.;
a = center to center of trusses in feet;
L = span of truss center to center of bearings in feet;
S = allowable average direct stress in lbs. per sq.in.

A more convenient approximation which will do for the usual cases is,

$$R=\sqrt{wa/S}\ \ (4L^2+60L) \quad . \quad . \quad . \quad . \quad . \quad (2)$$

From Art. 71, we have that the weight of purlins for one bay will be $8.5\ a^2\sqrt{wLn/S}$, where n is the number of purlins. We will make the coefficient 6 to allow for lower shop cost.

The weight per sq.ft. now becomes,

$$\sqrt{w/S}\ (6an^{\frac{1}{2}}L^{-\frac{1}{2}}+4a^{-\frac{1}{2}}L+60a^{-\frac{1}{2}}). \quad . \quad . \quad . \quad (3)$$

Making differential with regard to a equal to zero, we obtain minimum weight when,

$$a^3 = \frac{(L+15)^2}{9} \times \frac{L}{n} \quad \therefore \quad . \quad . \quad . \quad . \quad . \quad (4)$$

A number of important practical truths are not taken into account in the above discussion. There is a saving in the weight of the columns in spacing them farther apart, resulting, however, in heavier girts and crane girders. Efficiency of operation is often promoted by a longer bay than economy of first cost would dictate. Generally speaking, least cost of steel will be obtained by selecting a length of bay, about that given by above formula, such that the minimum weight of the shape to be used as a purlin will be loaded to its full capacity. The economical pitch of the roof is 6 to 8″ vertical to 12″ horizontal.

A very rough estimate of the cost of a framed steel building exclusive of land, filling, and foundations, can be obtained by adding together the total exterior surface and that of all floors which must be carried by the steel. 40 to 60 cents may be allowed per sq.ft., if no provision be made for cranes, and 60 cents to 1 dollar if there be such provision. Estimates for type (2) or (3) can be obtained by adding 40 cents per cu.ft. for concrete or brick work.

In the erection, one of two methods may be followed. The first is to rivet up on the ground each transverse bent (a roof truss and the columns on which it rests), the foot of the columns being placed near their foundations. These bents are then lifted up, one at a time, fastened in place, and then braced to those already erected by putting in position and finally bolting up the purlins, struts, bracing, and crane girders.

In the second and more common method, as each column is erected, it is braced by bolting up girders, struts, and bracing. If the roof truss were shipped in two or more parts, they are now riveted up in an horizontal position and hoisted on top of the columns, each being fastened to those already erected by purlins and bracing.

Derricks, gin-poles, and A-frames are common erection tools. A traveler is also used, consisting of a car on which is mounted a three or four-legged well-braced derrick.

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"Mill Buildings," by Tyrrell.
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Journal Western Society of Engineers; Vol. 13, p. 149.
Machinery, New York; Vol. 15, pp. 1 et seq.
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Costs

Engineering-Contracting, Vol. 35, p. 586.
Engineering News, Vol. 63, p. 96.
Engineering Record, Vol. 61, p. 126.

Prob. 123. Find approximate cost of a steel frame mill building, 100×300×30 feet to the eaves with a mezzanine floor 20 feet wide extending the full length of the building. Provision is to be made for a medium sized crane.

Art. 124. Outside Coverings

The purpose of the exterior covering is to exclude the water and weather, to retain the heat, and to protect from fire. The following table gives properties of various materials. Costs and weights do not include sheathing:

Type.	Weight in Lbs. Per Sq.ft.	Minimum Pitch.		Maximum Pitch.		Life, Years.	Cost, Cents Per Sq.ft.
		Bevel in 12″.	Angle with Hor.	Bevel in 12″.	Angle with Hor.		
Corr. Steel...	1 to 3	6	27°	Vert.	90°	3 to 10	5 to 10
Tin.....	1 to 2	½	2°	Vert.	90°	15 to 20	8 to 12
Slate.....	5 to 8	6	27°	Vert.	90°	25 to 30	8 to 12
Gravel.....	6 to 8	½	2°	3	14°	10 to 15	3 to 5
Slag.....	5 to 7	½	2°	3	14°	10 to 15	3 to 5
Tile	10 to 40	6	27°	.....	.....	Very durable	35 to 40

Of these corrugated steel and tile may be laid either with or without sheathing. Other coverings are usually so supported. All these types will exclude water and weather if properly designed. Corrugated steel alone neither retains the heat nor

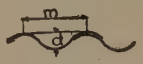

FIG. 124*a*.—Lap, 1 Corrugation.

FIG. 124*b*.—Lap, 2 Corrugations.

Corrugated Steel

protects against fire. The addition of sheathing overcomes the former objection. Slate will resist sparks but not extreme heat. All the remainder are good non-conductors and fire-resistant, tile being especially high in these qualities. On this account, it is considered our best roofing material and is much used for permanent structures in spite of the expense.

The sheathing which commonly supports these coverings is yellow pine, spruce, or hemlock, preferably planed on one side and matched. Span should not exceed 30 times its thickness.

CORRUGATED STEEL

Corrugated steel is made by crimping flat sheets as shown in Fig. 124*a*. *m* and *d* vary, the most common dimensions being $2\frac{1}{2}$ and $\frac{5}{8}''$ respectively. The usual width is 26″ after corrugation. Such sheets are readily obtained in lengths between 4 and 10 feet, but outside of these limits are subject to an additional charge. Bevel pieces and short lengths should be cut in the field, not less than 6″ in width being used. Common

FIG. 124*c*.

FIG. 124*d*.

FIG. 124*e*.

Methods of Fastening Corrugated Steel.

thicknesses are No. 18 and No. 20 U. S. gage for roofs; No. 20 and No. 22 for siding. When not supported by sheathing, maximum span is 5 feet for roofs and 6 feet for siding. Sheets should overlap longitudinally 4″ for siding and 6″ for roofs.

Transversely, it should have one corrugation lap for siding, Fig. 124*a*, and one and one-half corrugations lap for roofing. Corrugations run transversely to peak for siding and roofing.

Corrugated steel may be fastened to (*a*) sheathing, (*b*) wooden pieces called "nailing strips," bolted to the purlins, and (*c*) steel purlins. In all cases the nail, rivet, or bolt must pass through the top of the corrugation as shown in Fig. 124*c* to avoid leakage. To fasten to steel purlins or girts, we may use a clinch nail, $\frac{1}{8}''$ diameter, Fig. 124*d*, or steel straps, $\frac{3}{4}''\times$No. 18 U. S. gage, bolted with $\frac{3}{16}''$ stove bolts, Fig. 124*e*. In either event, fastenings should be spaced 8 to 12″ apart.

Special shapes seen in Figs. 124*f*, *g*, *h*, and *i* can readily be obtained. Sometimes moisture condenses on the inside of the roof and drops. Where this would be objectionable, place one or two layers of asbestos or tar paper just inside the steel and hold the same in place with poultry netting stretched from purlin to purlin.

Fig. 124*f*. Ridge Roll.

Fig. 124*g*. Gable Cornice.

Fig. 124*h*. Eave Cornice.

Fig. 124*i*. Outside Corner Finish.

The cost per pound of corrugated sheets painted black varies with the thickness, $2\frac{3}{4}$ cents for carload lots at Pittsburgh being an average figure. The same sheets when galvanized will cost $\frac{1}{2}$ cent per pound more.

Tin Roofing

This is made by joining together tin or terne plate. The body of either is charcoal iron, coke iron, or Bessemer steel in order of their value. These are coated with tin in the case of tin plate and lead for the terne plate, the former being preferable, especially where there are corrosive gases. These sheets are usually 14″×20″ or 20″×28″. The IC brand is the lighter and is preferred for roofing. IX is used for spouts, valleys, and gutters. They are fastened together by joints: parallel to the ridge, they are as seen in Fig. 124*j*; perpendicular, as in Fig. 124*k*. However, if the roof be flat, former must be

used in both directions. Tarred paper should be placed between the tin and the sheathing. Tin roofs are unsuitable for locations where they will be walked upon.

Slate Roofing

This is made by shingling a roof with thin pieces of slate,

Fig. 124*j*.
Flat Joint.

Fig. 124*k*.
Standing Joint.

Fig. 124*l*.
Slate Roof.

$\frac{1}{8}$ to $\frac{1}{4}''$ thick, 6 to 14″ wide, and 9 to 24″ long, laid as shown in Fig. 124*l*. The usual lap is 3″. Slates are fastened to sheathing by two nails, one in each upper corner. As slate will last much longer than steel, especially in the presence of corrosive gases, these nails should be of some durable metal such as copper, zinc, or composition.

It is particularly desirable here that sheathing should be matched and planed on one side. Tarred roofing felt between slate and plank helps to exclude weather and water. Another method of accomplishing this is to cement the joints.

Wooden I beam. Channel. Z bar. Angle.

Fig. 124*m*.—Forms of Purlins.

Gravel and Slag Roofs

These are made by laying and nailing several layers of felt on sheathing, and fastening them together with tar or roofing cement. The top layer is also coated with it to hold the gravel or slag on the roof. In *asphalt* roofs, asphalt is used for the cement.

Tile Roofs

Roofing tile may be of baked clay, terra-cotta, or reinforced concrete. They are of many sizes and shapes, usually having

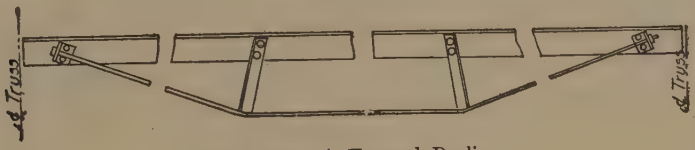

Fig. 124*n*.—A Trussed Purlin.

some interlocking device and a projecting lug on the inside by means of which they are fastened. They are laid much as shingles, either on sheathing, with or without roofing paper, or on purlins supporting each joint.

A roof may also be made of plaster on metal ribs in a construction similar to that employed for side walls in type 2*a* of preceding article.

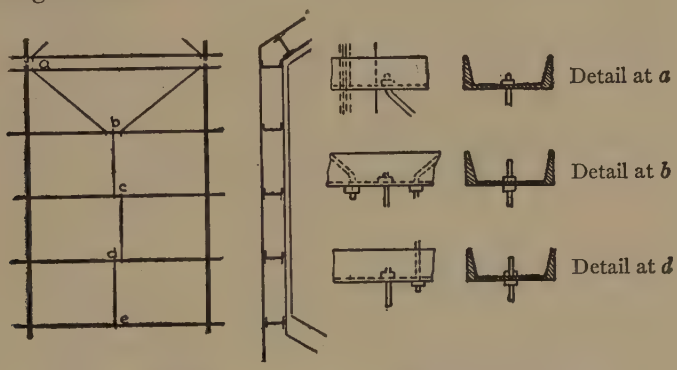

Fig. 124*o*.—Arrangement and Details of Sag Rods for Purlins.

Purlins and Girts

In addition to the types seen in Fig. 124*m*, we have the trussed purlin, Fig. 124*n*. Here the general form is that of the Queen post truss, Art. 30. A Z bar, one angle, or two angles may be employed for the top chord, single angles for the intermediate struts, and a riveted angle or a bent rod for the bottom tie. While the lessened weight may more than offset the additional shop cost, they are unsightly and lack lateral stiffness.

As field rivets are expensive, purlins sometimes connect to the clips by bolts. Fig. 124*m* gives the usual location of the nailing strips. They should be 2 to 3″ thick and bolted to the purlins every 3 feet by a $\frac{5}{8}$″ bolt. If the heads would interfere with the roofing, they may be countersunk in the wood. When purlins are more than 15 feet long, "sag rods," Fig. 124*o*, should be used to prevent deflection in the plane of the roof. These rods are $\frac{1}{2}$ to $\frac{3}{4}$″ diameter, threaded at each end to receive a nut.

Girts support the siding in much the same manner as the purlins carry the roofing. Typical forms with nailing strips and connections are seen in Fig. 124*p*. A $3''\times2\frac{1}{2}''\times\frac{1}{4}''$ is smallest sized angle, $3\frac{1}{2}''\times2\frac{1}{2}''\times\frac{5}{16}''$ and $4''\times3''\times\frac{3}{8}''$, larger dimension vertical, are typical.

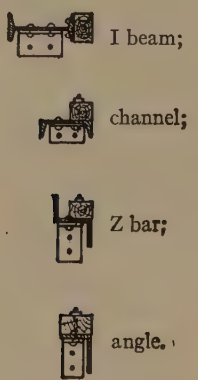

Fig. 124*p*.
Forms of Girts.

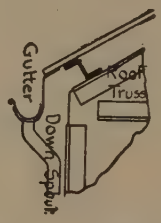

Fig. 124*q*.—Eaves.

Fig. 124*r*.
Flashing a Slate Roof.

Siding

In buildings of types (2) and (3), the masonry acts as siding. For type (1), there are four common methods of construction, all to be supported by girts as just considered.

(*A*) Corrugated steel.
(*B*) Corrugated steel on plank.
(*C*) Slate on plank.
(*D*) Plank alone.
(*E*) Cement on expanded metal, like type 2*a*, Art. 123.

The first three have already been considered under roofing. For (*B*) or (*C*), plank should be horizontal and fastened to intermediate nailing pieces, supported by the girts. In (*D*) plank should be vertical. It may be matched sheathing or plain lumber, battened, that is, having the cracks covered with small pieces of board about $\frac{1}{2}''$ thick.

Gutters, Spouts, and so Forth

At the intersection of the roof and the siding, the former is allowed to project from 6 to 24″, Fig. 124*q*, partly for looks and partly to catch the rain in a semicircular trough called the *gutter*. This and the downspout which carries the water from it to the ground may be designed by the aid of the principles of Hydraulics. Give the gutter a pitch of not less than $\frac{1}{2}$ of 1%, and a size sufficient when running full to carry 4″ of water per hour over the tributary area, this large amount being taken to provide a factor of safety. Proper thicknesses for galvanized steel are: No. 20 for valley * gutters; No. 24 for eave gutters and downspouts.

Many openings will occur in the roof and these must be flashed as shown in Fig. 124*r*, which gives an arrangement for an opening in a slate roof. Flashing may be of copper, lead, tin, zinc, or steel.

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Roofing, American Railway Engineering Association, Vol. 14, p. 839.
Down Spouts and Gutters, Engineering News, Vol. 47, p. 309.
See also references for Arts. 123 and 132.

Prob. 124. For Fig. 124*o* compute stresses in sag rods when bays are 20 feet; span of truss, 60 feet; and inclination of roof with horizontal, 30°. Loads are as follows: Roofing, 15# per inclined sq.ft.; snow, 12# per horizontal sq.ft.; wind, 25# per inclined sq.ft. Weight of purlin, 20# per lin.ft.

* The projection formed when two inclined roofs meet at a salient right angle is called a "hip"; the depression with a re-entrant right angle is termed a "valley."

Art. 125. Roof Trusses

As shown in Figs. 124*m* and 124*o*, the purlins rest on the top ord. Often they are put anywhere without regard to the tion of the joints. So placed, they cause large bending moments, which, when properly allowed for, require much more metal.

A better method is to locate them directly above the joints in the upper chord of the truss. Spacing of the purlins is usually 6 to 8 feet, seldom over 10 feet. Fig. 125*a* shows some of the most common types of trusses. Whether the loads are carried to the purlins by rafters as in wooden roof trusses, Art. 28, or roofing rests directly on the purlins, it is most economical to

FIG. 125*a*.

have the joints divide the upper chord into equal panel lengths. For long spans, they may be made like Pratt or Warren truss bridges. Either the Pratt or the Howe roof truss can be modified by giving it an appreciable depth at its supports as in Fig. 125*b*, perhaps lessening height at center. The plain Howe or Pratt roof trusses for spans 60 to 80 feet are not economical. The addition of a lantern for light and ventilation gives rise to forms like those seen in Fig. 125*c*. It will be noted that all three are statically indeterminate. The vertical member is usually assumed to be idle and the computation of the truss is thus rendered easy.

For a steel frame mill building, transverse bracing should be provided by extending a member downward at an angle of about

45° from lower chord joint nearest the end, Fig. 125*d*. This gives us partial bracing, Art. 99, and changes stresses in the truss.

The single angle (1), is objectionable on account of the eccentricity of its connection with other members. Hence it

FIG. 125*b*.
Modified Pratt
Roof Truss.

FIG. 125*c*.
Roof Trusses with
Lanterns.

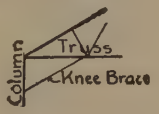


FIG. 125*d*.
Roof Truss with
Knee Brace.

should be used only for small tensile stesses. (2) is very good and is employed a great deal. For compression members, unequal legged angles with the longer legs in the plane of the truss are more economical since the same amount of material has a larger minimum radius of gyration. The outstanding legs are turned up for the top chord and down for the bottom chord. For the web members, the correct method is to turn the outstanding leg down for tension and up for compression, since stresses due to eccentricity and own weight are thereby partially balanced. In practice, however, these points are frequently disregarded to secure more compact connections. The washers, Figs. 37*k* and 125*e* (2), must not project from the angles. They should be spaced 18 to 24″ apart for compression members; 24 to 42″ for tension. (3) is usual where the top or bottom chord has to carry a load not at a joint. Though quite common, the projecting plate seems objectionable, espe-

(1) (2) (3) (4)

FIG. 125*e*.—Sections of Typical Roof Truss Members.

cially in compression, on account of a lack of lateral support. For this reason, it is best if the projection beyond the angles does not exceed 20 times the thickness of the plate. Rivet spacing should be about six inches and angles should be above for top chord and below for bottom chord. (4) is used only

for very heavy trusses and the construction is similar to that for riveted bridge trusses, Chapter VIII. For all truss members, the minimum thickness of metal should be $\frac{5}{16}''$, minimum leg $2\frac{1}{2}''$.

Although pin-connected roof trusses are built, they are not much used for mill buildings, and the riveted construction is more desirable. In Types (1) and (2), a plate large enough to hold the necessary rivets is provided at each joint. Rivets are generally $\frac{3}{4}''$ diameter, $\frac{5}{8}''$ being occasionally used in light work. Thickness of connection plates varies between $\frac{5}{16}''$ and $\frac{1}{2}''$, but is kept constant for a given truss.* With single angles the lower limit is preferred; with double angles, increasing thickness decreases number of rivets required and may be economical for that reason. The shape of the plate should be such that it may be cut from the rolled stuff with the least possible waste of material and labor. It is desirable that connections

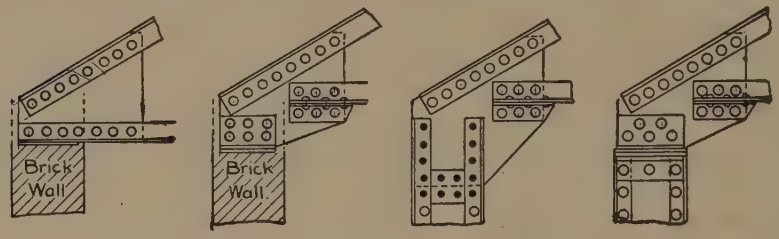


FIG. 125*f*. Incorrect. FIG. 125*g*. Good. FIG. 125*h*. Good. FIG. 125*i*. Good.

Methods of Detailing Roof Truss at Eaves.

be as compact as possible and that each outside rivet be located near but not too near the edge, Art. 50*e* (1). With Type (3), the plate itself serves to connect the chord with the web members. In Type (4), two connection plates are used, one inside of each channel. Fig. 132*h* gives several examples.

Two rivets are the least which should be used in a connection. Wherever stresses call for more than four rivets, a lug angle is preferable, thus making a more compact connection and reducing the eccentricity. Center lines of pieces should intersect at a point, but in practice the usual method is to make

* It may, however, be made greater at joints where there are heavy stresses, thereby lessening number of rivets. This will cause no particular trouble in the shop unless there be a cover plate on the angles.

rivet lines of members meet, since it simplifies work in pattern shop and drawing room. This principle is usually followed except at the ends, where the common detail is as seen in Fig. 125*f*. Much better but more expensive arrangements are shown in Figs. 125*g*, 125*h*, and 125*i*.

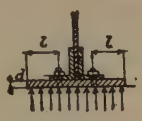


Fig. 125*j*.
Masonry Plate for a Roof Truss.

In a building with masonry supporting walls, an unplaned plate, not less than $\frac{1}{2}''$ thick, is riveted on the bottom. This plate must have an area such that the maximum unit pressure on masonry is within permissible values, Art. 131. The thickness, d, for any given projection, l, can be obtained by treating it as a cantilever beam. The bolts are usually about two in number and $\frac{3}{4}'' \times 20''$ or thereabouts.

For an expansion end, the above named sole plate must have slotted holes and be planed on its lower side. Underneath it is located the wall plate, of the same thickness but planed on its upper side. It is made of the same width or a little wider, somewhat longer to allow for expansion and the bolt holes are not slotted. For long spans, say those over 75 feet, roller bearings, either with or without pins, Art. 61, are employed. As already noted, this provision for expansion is often omitted.

REFERENCES

Saw Tooth Roofs

Eng. News, Vol. 56, p. 627.
Eng. Rec., Vol. 54, p. 626.
See also references for Arts. 123 and 132.

Prob. 125. Consider in the example of Art. 132, that a wind load of 25 lbs. per vertical sq.ft. is applied horizontally to roof as well as sides. What member will be most affected by the change? Next answer same question with wind loads equal to those given in Art. 132 but applied vertically. Note in the latter case that the knee brace is supposed to be out of action. Consult last paragraph of Art. 28.

Art. 126. Columns

The outside columns usually have small loads but a large moment transverse to the building. To meet these conditions, the I-beam column, either rolled or built up, the H section, or latticed channels are best. The first is preferred where the bending moment is an important factor. Place webs in plane of maximum moment. For very light transverse forces, the four angles of the built I may be latticed together, batten plates being placed at each end, at the knee brace, and at any other heavy loads.

The outside face of columns for buildings of Type (2) should be at least one inch within the masonry wall. Inside, the column may project or pilasters be built around them. In brickwork, design column and wall so that brick will need as little trimming as possible. An average brick with its share of the mortar joint will "lay" 8″ long by 4″ wide by $2\frac{1}{2}$″ deep or perhaps a little more. Thus a 16″ wall means one where there are four courses of brick; however, it is very likely to measure something more and many engineers term it a 17″ wall. Thickness of solid masonry walls should not be less than six inches plus one-fiftieth of the height.

Interior columns seldom carry an appreciable moment. Compactness and economy are the main considerations leading to the selection of the H section, latticed channel, and Z bar types. Columns which have light crane loads are designed like those outside, of which we have already spoken. With heavy loads, a stiff column, capable of carrying considerable moment in either direction becomes a necessity. These stresses are well provided for by the two channel and I section, that of two channels latticed being weak against bending in the plane of the latticing. The crane girder is then placed directly over the center of the column and knee braced to it in a longitudinal direction.

There are three methods of providing for the very frequent case where the same column acts as a support for the crane girder and the roof truss, Figs. 126*a*, 126*b*, and 126*c*. The favorite sections are the built I, two latticed channels, and the two channel and I.

A typical cap is shown in Fig. 126*d*, which gives a design for the top of a two-channel column. The plate is $\frac{1}{2}$ to $\frac{3}{4}''$ thick and unplaned. It is made only large enough to cover column section and projecting angles. Channels should be milled off

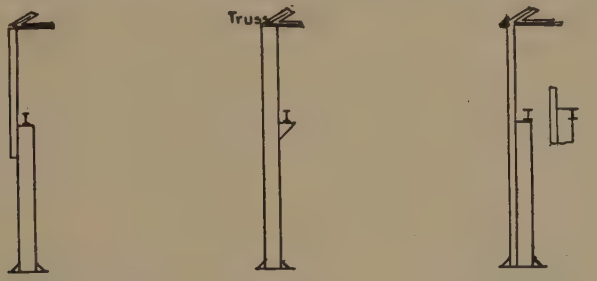


FIG. 126*a*. FIG. 126*b*. FIG. 126*c*.

Combined Crane and Roof Truss Columns.

after they are latticed and then angles placed flush. This ensures a bearing that will carry the full capacity of the column. Hence but few rivets are required, just enough to tack down the plates and angles. However, if tops are not milled, horizontal rivets must be computed for full vertical load.

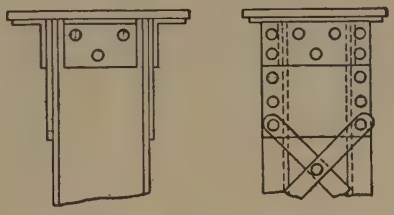


FIG. 126*d*.—Column Cap.

CONNECTIONS

It is desirable that connections be economical of shopwork, easy of erection, and that they bring loads as close to the center of gravity of the column as possible. They may be classified as:

(1) *Shelf Angles.*
(2) *Connection Angles.*
(3) *Brackets.*

(1) *Shelf Angle Connections.*

Fig. 126*e* shows the connection of a girt to a built I column and Fig. 126*f* gives joint of a laced channel section with an I beam. In the latter, the upper angle may be omitted if the I be otherwise stiffened. The horizontal rivets in the upper

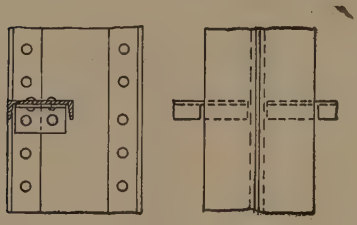


FIG. 126*e*.

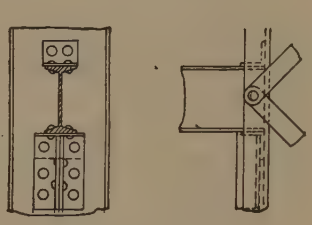


FIG. 126*f*.

angle are often made field for ease of erection. They should not be considered to assist in carrying the load because of the weakness of the outstanding leg in flexure, and the fact that the rivets are already stressed in tension. This type is easy to erect but does not stiffen the web like (2).

(2) *Connection Angles.*

FIG. 126*g*.
Connection Angles.

FIG. 126*h*.

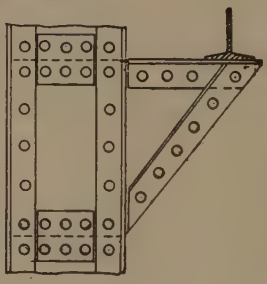


FIG. 126*i*.
Bracket.

Fig. 126*g* shows a connection of this kind between a two-channel and I-beam column and a plate girder, while 126*h* gives that between a column of two latticed channels and an I beam.

(3) *Brackets.*

These may be used either to act as a brace or to support a member. The latter case occurs when a crane column carries a load above the crane and the arrangements shown in Figs. 126*a* and 126*c* are undesirable. Fig. 126*i* shows a bracket as designed for a built I column. It will be noted that the web is broken at the bracket. Both joints should be milled. In small brackets, the diagonal angles are sometimes omitted. However, this should not occur when the diagonal length is greater than thirty times the thickness of plate except for very light loads. For other cases, diagonal angles should be designed

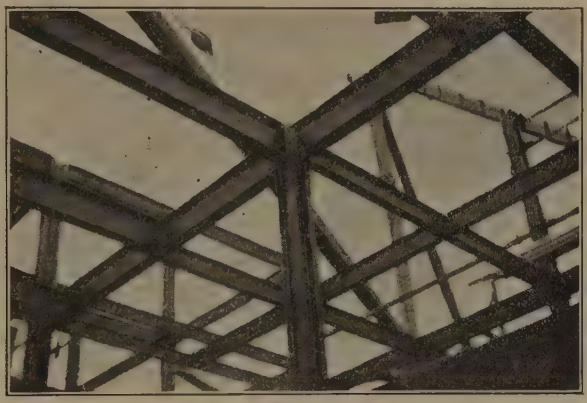


FIG. 126*j*.—Framing in Municipal Asphalt Plant, Dallas Ave., Pittsburgh, Pa.

for their stresses and fitted at upper ends. They must contain rivets enough for their loads unless they are fitted at lower end.

BASE

Design of the base should correspond with assumptions in computation. In any event, provide enough area that allowable stress on masonry is not exceeded, and bolts sufficient for any possible uplift or shear. If the columns have been assumed to be hinged at base, the latter should have no more than the necessary bearing and the bolts placed near the neutral axis with regard to the principal bending moment. If this end has

been assumed to be fixed, base, bolts, and masonry must be designed to carry the moment or moments from the column into the soil.

For the column free at the end, bolts $\frac{1}{2}$ to $1\frac{1}{4}''$ in diameter

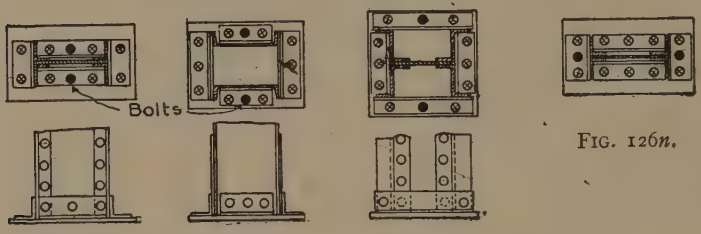


Fig. 126*n*.

Fig. 126*k*. Fig. 126*l*. Fig. 126*m*.

Column Bases.

are used. If plain, they should extend into the masonry about 30 diameters. The bolt may be ragged, swaged, of the expansion or U type, or have a plate washer, Art. 46. Figs. 126*k*, 126*l*, and 126*m* show typical, free-ended, column bases.

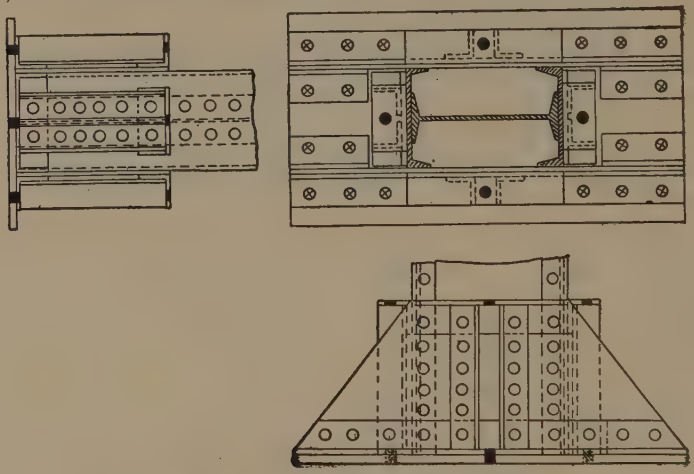
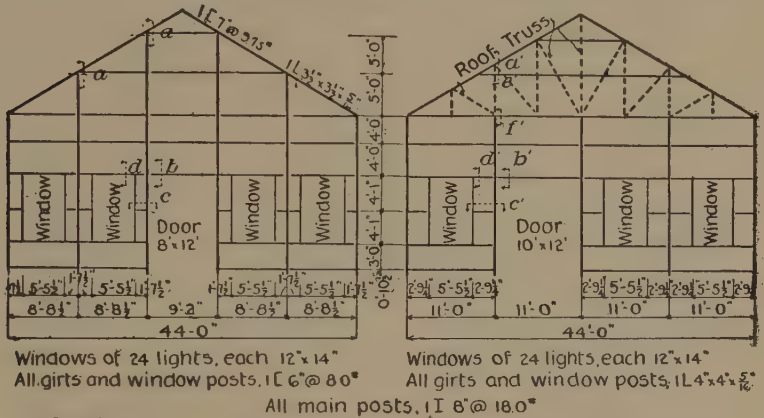


Fig. 126*o*.—A Fixed-ended Column Base.

Such a column, however, must depend entirely on the bracing above. Even if this provides sufficient strength, bent is stiffened by making the end partially fixed. But care must be taken that no injurious stresses are caused thereby, that is,

that the column is fitted to resist the forces caused by this partial continuity. For example, Fig. 126*k* is much improved by a detail as shown in Fig. 126*n*. In this case, a solid plate should



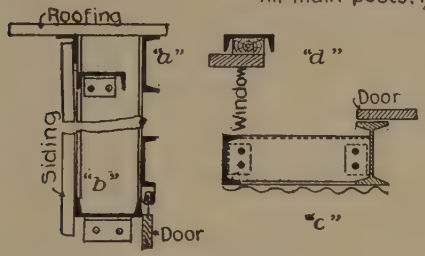


FIG. 126*p*.—Gable Rafter Construction.

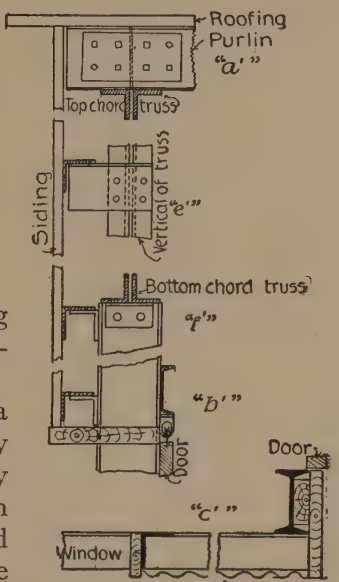


FIG. 126*q*.—Trussed Construction.

be used to unite the angles, as lacing is insufficient to resist a considerable bending moment.

For a column fixed at the end a much larger base area is necessary in order that the bolts shall carry the tension and that the stress on the compression side shall not exceed the safe bearing strength of the masonry. Fig. 126*o* shows a base for a fixed-ended column with a two-channel and I-beam section. Stiffeners under shelf angles for bolts must be fitted at both ends. As columns, they are

computed to carry loads on the bolts. Also sufficient rivets must pass through them to take these loads into the main section. Gusset plates ought to be at least $\frac{7}{16}''$ thick and their angle with the vertical should not exceed 45°. Instead of these details, expensive in shop work, the milled end of the column may rest on a single heavy plate to which it is fastened by light angles.

Gable Ends

When buildings may be extended, end roof trusses and columns should be made the same as intermediate, Figs. 126*q*. The siding is carried by girts, as already considered in Art. 124, ease of removal rather than permanency being an advantage. These girts are carried by columns extending to truss, occasionally to its top chord. If built I's are employed, the web should be transverse to the siding and the two outer angles only extend across the truss.

When end is permanent, the same construction may be used. It is possible that some saving may be effected by lightening columns and trusses due to the lesser loads. Consider, however, additional shop work as well as decreased weight. If economy is sought, a better method is to use "gable rafters," Fig. 126*p*. Here beams, usually channels, figured to carry the reactions from the purlins, are supported by columns which carry the siding girts.

Prob. 126. Design a column 20 feet high, unsupported at top, to carry a load of 120,000# which may be inclined at an angle of 10° with the vertical in any direction, using allowable stresses of Art. 131. Material to be soft steel.

Art. 127. Bracing

A steel building should be efficiently braced at every point. A masonry wall will stand the wind loads in its own plane, or, if designed as solid, is sufficient for moment caused by forces perpendicular thereto. It also sometimes withstands other horizontal loads of more or less severity; but steel bracing is much better, especially if the loads be heavy or suddenly applied. Roofing, siding or flooring is frequently supposed

to provide rigidity, but either of these assumptions should be used with a great deal of caution. Generally speaking, the systems necessary to properly stiffen a building, are:

(*a*) Bracing in the plane of top chord of roof truss.

(*b*) Bracing in the plane of bottom chord of roof truss.

(*c*) Bracing in the side of the building.

(*d*) Bracing in the plane of roof truss.

We may add still one more, although it hardly seems needed if (*a*) and (*b*) have been properly designed:

(*e*) Bracing in a vertical plane through peak of roof.

Interior columns are often braced by their connection to points which are panel points in above systems; or by the depth

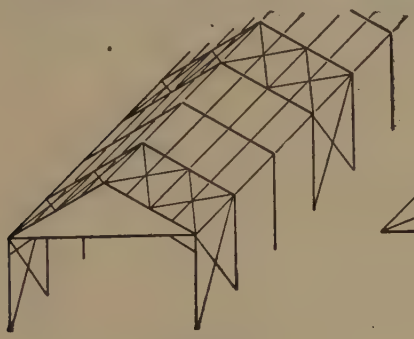


FIG. 127*a*.
Diagram of Bracing.

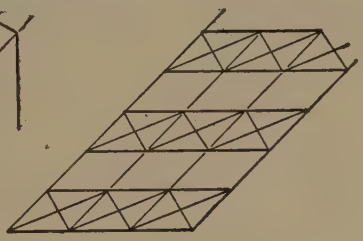


FIG. 127*b*.
Bracing for Bottom Chord.

of the beams to which they connect in addition to the stiffness of the columns themselves.

Bracing may be classified according to the design of its members as in Art. 96.

(*1*) *Bracing with adjustable members.*

(*2*) *Bracing with stiff tension members.*

(*3*) *Bracing with compression members.*

We will also add another type from Art. 99.

(*4*) Knee bracing.

(4) is used only where a full system is impracticable. (1) is the usual style, (2) is much better, and (3) is the best of all the expense increasing with the quality. Bracing with compression members is seldom used in mill buildings, and only on heavy work.

Bracing is not necessary in every bay. It is customary, however, to put knee braces in the plane of every truss. There should be X bracing in the plane of bottom chord, Fig. 127*b*, in about every other bay, and for the top chord and sides in every third bay. In the latter case, the purlins and girts which serve as struts transmit the support given by the bracing to those points with which it is not in contact. In case the bottom chord bracing is spaced more than two bays apart, struts must be inserted to stiffen the intermediate trusses, Fig. 127*a*.

FIG. 127*c*. FIG. 127*d*.
Lateral Lugs.

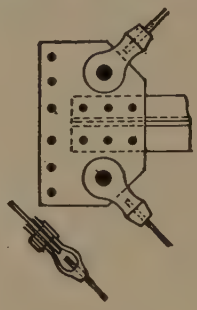


FIG. 127*e*.
Lateral Plate.

Besides the loop rod, Art. 42, and the clevis nut, Art. 43, the methods shown in Figs. 127*c* and 127*d* are very common. The latter can be used for almost any angle. The projection, *a*, engages a hole in the web and transmits the stress into the strut. In the bracing in the plane of the top chord, a single light angle or a $\frac{3}{4}''$ round, upset to $1''$, is the common composition of each diagonal. Purlins are the usual struts, some engineers insisting that they be riveted at the braced panel. With the nut and washer connection, the slotted hole necessary for same is placed in connection plates just below the angles. For a single loop rod, double lateral plates must be provided. With a clevis or double loop, one is required as detailed in Fig. 127*e*.

Bottom chord connections are similar to those for the top chord. Somewhat heavier angles are found here and the use of rods is less common, $\frac{7}{8}''$ upset to $1\frac{1}{4}''$ being an average size. The plate should be attached to the bottom chord, preferably to both angles. The bracing may be in the form of Warren or Pratt trusses. In the former case, members capable of

carrying compression must be used, two angles riveted together being common. For the latter, one single angle constitutes each of the two diagonals in a panel, while two angles, a channel or an I-beam serves as the strut.

Bracing in the sides and ends should be placed as near the center of the columns as practicable. This may be readily arranged with the nut and washer fastening, but the lateral plate attached to the back of the column produces an objectionable torsion.

At the eaves where all these systems of bracing intersect is placed the eave strut. It should never be omitted in a steel frame building. A channel is sometimes used, but two pairs of angles, about two feet deep back to back of angles and latticed together, are better.

REFERENCES

Wind Bracing

Journal of Western Society of Engineers, Vol. 17, p. 987.
Eng. News, Vol. 53, p. 99.
See also references for Arts. 123 and 132.

Prob. 127. A rectangular upright framework, 20 feet square, Fig. 127*f*, carries a compression on each vertical member of 150,000 lbs. and a horizontal force acting on the outside in either direction of 40,000 lbs. Design in soft steel for four different types of bracing and compare costs. Make main members of two channels. Use specifications of Art. 131. Steel, 4 cents per pound.

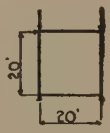


Fig. 127*f*.

Art. 128. Floors

The first floor is laid directly on the ground. Great pains must be taken to drain surrounding soil and to keep out surface water. The following floors may be used:

(1) Dirt. (3) Asphalt. (5) Wood.
(2) Concrete. (4) Brick.

(1) *Dirt Floors*

These are made by excavating the loam or unstable ground to a depth of say one foot, ramming and refilling with gravel, cinders, slag, foundry loam, or other suitable material. In case the natural soil is stable, it may be simply rammed. These floors are cheap and serve well for foundries, rolling mills, structural shops, and the like.

(2) *Concrete Floors*

Excavate all material to 5″ below finished surface; all ground not perfectly solid to be removed for 7″ deeper and refilled with broken stone, gravel, or cinders, thoroughly tamped. Upon this is to be laid a base of 1 part Portland cement, 3 parts clean sand, and 5 parts broken stone. Last comes the wearing coat of 1 part Portland cement and 2 parts $\frac{1}{4}$″ screenings of sound stone. There are slight variations in the last layer, one of which is called the "granolithic."

(3) *Asphalt Floors*

In the preceding, substitute for the wearing surface a coat of asphalt about 1″ thick. This should be not used where oil will drip and destroy it. It is more elastic and more agreeable to the feet than concrete.

(4) *Brick Floors*

Brick may be laid flat or edgewise on a one to two inch layer of sand upon a well compacted soil, or, occasionally, a concrete base. Either tar, sand or cement may be used for the joints.

(5) *Wooden Floors*

The best wooden floors have a surfacing of $\frac{7}{8}$″ matched maple flooring resting on plank at right angles to the same, which in turn is nailed to sleepers about 6″×6″, as shown in Fig. 128. The latter may be laid in 6 to 8″ of concrete, gravel, or cinder filling. Timbers will be subject to dry rot unless protected by lime mortar or coal tar. Sometimes planks rest on a base of coal tar concrete.

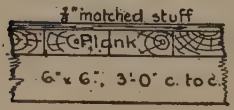


FIG. 128.—Wooden Floor.

Upper Floors

Systems described in the following chapter are sometimes employed; in this article, we will mention a few types whose use is confined to mill buildings.

As on the first floor, wood makes the best surface to work on; its chief disadvantage is the danger from fire. This is largely overcome by the use of the slow-burning construction, the idea being to make the timbers so large that they burn slowly and afford opportunity to fight the fire. This construction consists in spacing the wooden beams eight or ten feet on centers, and making the plank thick enough to carry the load for that span. The more common type, top flooring of $\frac{7}{8}''$ matched maple on under flooring of rough boards, laid at an angle of 45 or 90° with the preceding, is sometimes seen in mill buildings. It is supported on joists, spaced two or three feet apart and "bridged" at intervals not exceeding ten feet. For either construction, the joists may be I-beams with nailing strips. The Hy-Rib construction in the form of an arch may also be employed.

Where an incombustible floor is a necessity, steel plates riveted to I-beams may be used. However, they lack stiffness and are slippery. Buckled plates, Art. 42, covered with three inches of concrete make a solid floor to which these objections do not apply.

REFERENCES

Eng. News, Vol. 40, p. 271.
Eng. Rec., Vol. 61, p. 693.
Eng. Mag., Vol. 43, pp. 567, 710.
Iron Age, Vol. 87, pp. 378, 426.
See also references for Arts. 123 and 132.

Prob. 128. Compare feet B. M. of rough boarding on joists 30″ c. to c. with a plank floor. Spans 6, 8, and 10 feet. Live load, 150 lbs. per sq.ft. Allowable flexural stress, 1200 lbs. per sq.in.

Art. 129. Cranes

We shall give here only enough data for the design of the structural part of the building. For the details of the cranes themselves, see Vol. III. Two types of traveling cranes are common in mill buildings; the bridge, Fig. 129*a*; and the jib, Fig. 129*b*.

Bridge Cranes

Let W = capacity of crane in pounds;
S = span of crane feet;
P = maximum load on each wheel in pounds.
B, C, and D are distances shown in Fig. 129*c* in feet. As a very rough idea until exact dimensions can be obtained, we may use,

$$B = 4 + \frac{W}{10{,}000},$$

$$C = 0.6 + \frac{W}{200{,}000},$$

$$D = 4 + \frac{W}{20{,}000} + \frac{S}{9},$$

$$P = (W + 10{,}000)(0.4 + .006S).$$

The rail on which they run should be usual railroad-rail the A.S.C.E. (American Society of Civil Engineers) being the standard. They should be specified to weigh not less than one pound per yard for each thousand pounds of maximum wheel load. These rails may be bolted on through their flanges, hook bolted through their webs, or fastened by clips as in Fig. 129*d*.

Beams which support the rails are I-beams either rolled or built. In either case, they are made much like those in Chapter VI, although some changes are caused by the heavy concentrated loads which they carry. As previously demonstrated, the economic depth for built sections in the absence of other limitations, is that which makes the weight of the web equal to that of the flanges. Since the two girders which support the crane

FIG. 129*a*.—The Bridge Crane.

FIG. 129*b*.—The Jib Crane.

cannot brace one another, the top flanges must be computed as struts and their allowable stresses reduced by a compression formula. At the end they must be stiffened laterally. If the columns stand alone, this may be done as seen in Fig. 129*f*.

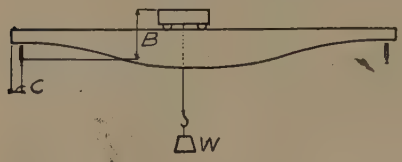


FIG. 129*c*.
Diagram of a Bridge Crane.

FIG. 129*d*.—Fastening for Crane Rail.

For supports like Figs. 126*a*, 126*b* and 126*c*, the girder may be fastened to the upper part of the column by a detail similar to Fig. 129*g*. The girders are preferably milled and firmly fastened together to make them continuous and thus add

FIG. 129*e*.—Shop of Midvale Steel Co., Philadelphia, Pa.
McClintic-Marshall Construction Co., Rankin, Pa.

rigidity. At points not over 100 feet apart, insert expansion joints by putting slotted holes in one end of one girder at all points where it is fastened to the column and to the other girder.

Provision for shears and concentrations is especially important here. Emphasis may be laid on figuring end stiffeners for maximum shear, intermediate for maximum wheel load, and rivets

FIG. 129*f*.

FIG. 129*g*.

Stiffening a Crane Girder.

FIG. 129*h*.

FIG. 129*i*.

Support for Jib Cranes.

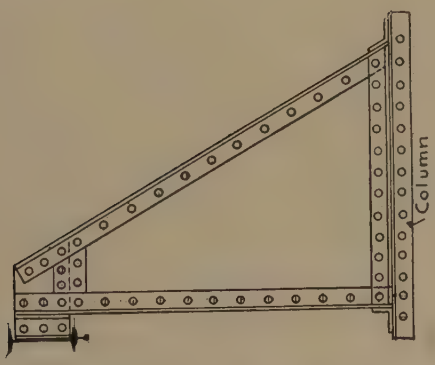


FIG. 129*j*.—Support for Upper Wheel of Jib Crane.

in top flange for concentration distributed over 24 to 36″, varying with the size of the rail. There is nothing new about the make up of the flanges except that channels may be used for top cover plates on account of the added lateral stiffness. Girders should be knee braced to the column to resist tractive forces.

Jib Cranes

In a jib crane, the upper reaction is horizontal, while the lower one is inclined. The latter may be taken, either by two wheels, one each for horizontal and vertical components, or by one wheel designed for both. In the first case, an I-beam underneath each rail, as shown in Fig. 129*h*, may be used. When only one rail is employed, it can be designed as in Fig. 129*i*. The upper rail is carried as shown in Fig. 129*j*.

See Vol. III for references and a further discussion of cranes.

Prob. 129. In Fig. 129*c*, let the length of bay be 18 feet; *P*, 15,000 lbs.; *D*, 10 feet. Design crane girder. Use specifications of Art. 131.

Art. 130. Windows, Skylights, Doors, and Ventilators

Illumination

Plenty of natural light in a building is very desirable, as artificial substitutes are poor and costly. Direct sunlight is an excess that is quite objectionable, the ideal illumination being uniformly distributed. From 20 to 40% of the surface of a building should be windows, the amount depending upon the kind of work carried on within. Thus a blacksmith shop requires far less than a machine shop. A portion of this light, especially for wide buildings, should be from skylights. Both windows and skylights are expensive, require more care, and are much less weather proof than the rest of the building.

Glass

Glass or transparent fabric may be used to admit the light. Ordinary glass may be $\frac{1}{16}''$ thick, "single strength"; $\frac{1}{8}''$ thick, "double strength"; or $\frac{1}{4}''$ thick, "plate glass." Of the first

Fig. 130*a*.—Ribbed Glass.

Fig. 130*b*.—Prism Glass.

two, there are three grades: "*AA*," best; "*A*," good; and "*B*," poor. "*A*" should be used for mill buildings. The only kind of plate glass employed is that which is not fit for other purposes. Ordinary sheet glass passes the rays directly

through it. To give a better distribution and at the same time avoid the direct sunlight, we may use:

Ribbed glass, Fig. 130*a*.

Figured or hammered glass, that is, glass with one or both surfaces so broken up as to diffuse the light.

Prism glass, Fig. 130*b*.

These so split up the light rays that we cannot see through them. This is an advantage in some locations.

Glass has two grave faults—it is a weak point in many buildings otherwise efficiently protected against fire, and, in breaking and falling, it is dangerous, particularly in skylights. These objections can be met by using wire glass, that is, glass in which wire is embedded. Any of aforementioned kinds may be thus treated. These different varieties of glass are not only more expensive, but also more difficult to replace, as common glass can be obtained almost anywhere.

Translucent fabric is a patented wire cloth which is used as a substitute for glass. It is elastic and does not break; it is moderately fire resisting; it passes less light than glass, but does not require shades; its cost, tacked on the sides of a wooden frame, is about the same as glass.

Glass is sold in rectangular sheets, the cost per square foot increasing with the area. Design plant to use as few sizes as possible and only such as hardware dealers carry regularly in stock. Although odd sizes can readily be procured, the charge is for the standard sheet from which it is obtained plus the cost of cutting. $12'' \times 14''$ is a good size, partly because it is the largest in which single strength glass should be used.

Windows

There are a number of types, of which the principal are:

Fixed, usually of one sash which is immovable.

Sliding, where the sash slide horizontally.

Swinging, mounted on a horizontal pivot through the center.

Weighted, usually of two sash, balanced by weights in boxes at the sides. This is the usual dwelling-house window.

Balanced, also of two sash where one window counterbalances the other.

Fig. 130*c* might be either balanced or weighted. It shows a

window made of two sash, each of 6 lights 12″×14″. From dimensions given, size of other sash and windows may be readily computed. These sash set in a frame approximately 2″ thick which must be fastened to the girts. At the bottom, it has a shape about as shown in Fig. 130*d*. Parts of the frame are often omitted for fixed sash. For weighted windows, the side pieces will be about 5″ thick. In a brick or concrete building, the frame should be nailed to 3″×4″, or thereabouts, set in the masonry. Cost of windows with frames is about 40 cents per sq.ft.

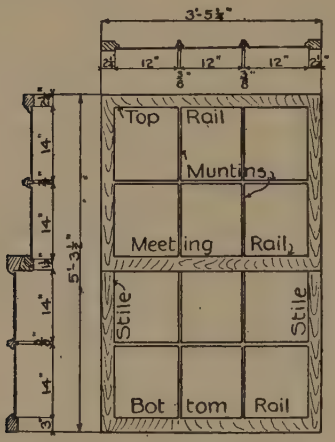


FIG. 130*c*.—Sash Dimensions.

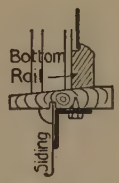


FIG. 130*d*.
Section of
Window Frame.

SKYLIGHTS

These are windows set in the roof. They are usually placed a little above the general level to avoid leakage. Fig. 130*e* shows a skylight with a metal frame. Note the arrangement in the detail of the sash bar by which the water of condensation is collected. A wooden sash bar has an equivalent device, Fig. 130*f*.

Glass should be $\frac{1}{4}$″ thick. Two or more sheets may be put in without the necessity for transverse sash bars, the upper sheets lapping over the lower. Unless translucent fabric is used, a wire netting should be stretched underneath to catch the falling glass. Skylights, complete, cost about 60 cents per sq.ft.

Doors

These should be so located as to allow the economical operation of the plant. At least one door must be big enough to permit the largest car, wagon, or other article to enter or leave preferably without change. Do not forget engines or other equipment which will arrive after the covering is on, or which must be replaced at frequent intervals. Doors, however, like windows, admit the weather and are expensive to build and maintain, costing about 40 cents per sq.ft. It might be

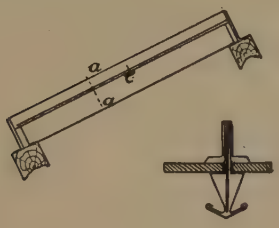


Fig. 130*e*.
Metal Skylight.

Section *aa*.

Fig. 130*f*.
Detail of Wooden Sash Bar.

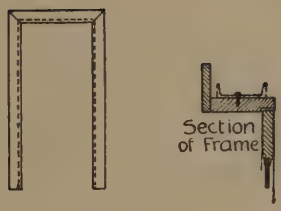


Fig. 130*g*.—Door Frame.

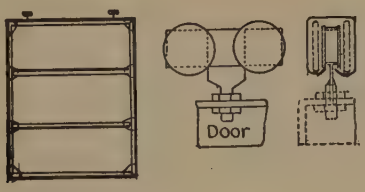


Fig. 130*h*.—Sliding Door.

argued that this is not much less than for a brick wall. But it should be remembered that the total cost of a brick wall is little changed by a small opening. There are three types:

Swinging.
Sliding.
Lifting.

Swinging doors should be made $\frac{1}{4}''$ less than the door frame. The latter, Fig. 130*g*, should be at least $1\frac{3}{4}''$ thick and must be

fastened either to the steel work or a bricked-in nailing piece. Do not use less than $2\frac{1}{2}\times6\frac{1}{2}$ feet except for cupboards. 3×7 is a good size and is usually carried in stock. Openings less than 6×9 feet high may be filled by a single-swing door; up to 12×9 feet by a double door. For small doors, use butt hinges; for large doors, strap hinges. The latter should generally be arranged to swing outward.

Sliding doors should overlap frame 2″ top and sides. It may be supported from hangers and wheels running upon a track, one scheme being shown in Fig. 130*h*. At the bottom of posts there should be bent guide plates to prevent the door from blowing in.

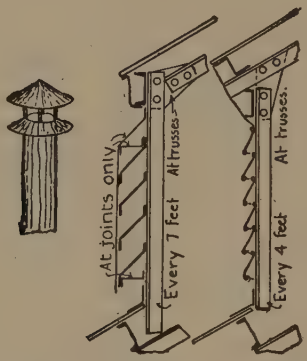


FIG. 130*i*. Ventilator. FIG. 130*j*. Shiffler. FIG. 130*k*. Berlin. Louvres.

Lifting doors are nearly all large and may be raised bodily or rolled up. They are often provided with a small swinging door to allow a single man to enter.

VENTILATORS AND LOUVRES

For purposes of ventilation, provide for every 100 sq.ft. of floor space, about $\frac{1}{2}$ sq.ft. net opening of ventilators for machine shops, 6 sq.ft. for mills, and 8 sq.ft. for forge shops and foundries. For louvres, increase above amounts by $\frac{1}{2}$.

There are many varieties of ventilators, Fig. 130*i* showing a typical one. The idea is to allow the warm vitiated air to leave the building without permitting the rain to enter.

A louvre is an arrangement of fixed steel slats, usually placed in the vertical sides of monitors, see Figs. 130*j* and 130*k*.

REFERENCES

Eng. News, Vol. 40, p. 391. Glass.
Eng. News, Vol. 45, p. 33. Glass.
Eng. News, Vol. 54, p. 94. Lighting.
See also references for Arts. 123 and 132.

Prob. 130. Estimate roughly the cost of providing light for a machine shop 150 feet long, 80 feet wide, and 25 feet to the eaves.

Art. 131. Loads and Specifications

There is a great deal of light mill building work. Many owners seek competitive prices without any engineering advice other than that tendered them by the contractor. Our standpoint has been the best practice for all concerned, which means such an investment in the building that it will be in all respects safe yet make total annual cost a minimum.

A legitimate reason for light work arises in temporary buildings or in those where frequent vital changes in the industry are taking place. In this case, it is not wise to enforce those precautions whose sole value is to prolong the life of the structure.

We will give here simply the points in which specifications for mill buildings differ from those for railroad bridges. While we refer to Art. 50 of Vol. I, we will so treat the subject that changes can be applied to any specifications.

SPECIFICATIONS FOR MILL BUILDINGS

Omit (*a*), *Description*, Art. 50.
Rewrite (*b*), *Loads*, as follows:

(1) DEAD LOADS. Dead loads assumed in computation must be checked from working drawings.

(2) SNOW LOADS

The snow loads are determined by the formula,

$$w = (l - 25) \cos i$$

where w is the weight in lbs. per hor. sq.ft., l is the latitude in degrees, and i is the angular elevation of the roof, also in degrees. (This amount may be increased for high altitudes and decreased for arid lands. It will vary with the heat in the building and with the care taken to prevent overloading by snow.)

(3) WIND LOADS

The pressure of the wind will be computed by the formula,

$$p = C 2 \sin \theta / (1 + \sin^2 \theta)$$

where p = pressure in lbs. per inclined sq.ft. normal to the surface, θ = angle of inclination with the horizontal, and C is a constant, varying somewhat with location and exposure. Details shall be designed to take care of a pressure from within of C lbs. per inclined sq.ft. (The latter clause is intended to take care of the negative pressure sometimes occurring in a violent storm, or of the excess inside pressure caused by the wind blowing through openings on the windward side. The proper value of C for ordinary locations, used as in these specifications without increase of allowable stresses, is 25. This should be doubled or tripled for regions subject to earthquakes or hurricanes. Some authors prefer a larger wind load and increased allowable values for combinations including wind as we have already done for railroad bridges.)

(4) CRANE LOADS

The maximum axle loads for the cranes as obtained from the manufacturers shall be increased 25% to allow for impact. Provision shall also be made for horizontal forces equal to 10% of actual axle loads, acting either along the tracks or transverse to them.

(5) FLOOR LOADS

Probable live load on the floor should be computed, never being taken at less than 100 lbs. per sq.ft. Where a considerable portion of the load is machinery, a suitable allowance is to be added for impact, the minimum total being 150 lbs. per sq.ft.

(6) MISCELLANEOUS LOADS

(Each building has its own individual conditions which must be investigated and loadings given which will ensure proper provision to meet these stresses. Trusses often support temporarily tackles for lifting boilers or machinery underneath them. Floors may need to be investigated for the transportation of some equipment or other part of the the building. Erection should sometimes be considered. In regions subject to earthquakes, avoid masonry walls, use efficient connections, and design for high wind stresses as mentioned above.)

(*c*) *Allowable Unit Stresses in lbs. per sq.in. for steel.* (Replacing (*c*) of Art. 50.)

(1) ALLOWABLE UNIT STRESSES

Let us take E to represent the safe unit strength for quiescent tension, equals 14,000 for soft steel and 16,000 for medium steel. Further let R equal the maximum slenderness ratio equals unsupported length in inches divided by the radius of gyration also in inches.

Tension on net section	E
Compression on gross section,	$E(1-.004R)$
Shear on shop rivets, pins, and gross sections of webs	$2E/3$
Shear on field rivets and bolts	$E/2$
Bearing on shop rivets and pins	$4E/3$
Bearing on field rivets and bolts	E
Flexural stresses	E

(2) COMBINATIONS — Maximum possible combination of loads shall not cause stresses exceeding allowable values except that an excess of 30% may be permitted for the simultaneous action of the wind and the horizontal crane load.

(3) OTHER ALLOWABLE STRESSES — For wood, see Art. 6. Allowable compression on wall masonry in lbs. per sq.in. is as follows:

Brickwork, lime mortar	100
Brickwork, Portland cement mortar	150
Concrete, 1 Portland cement, 3 sand, 6 stone	250
Stone masonry, first class	250

These values may be increased 50% for local bearings on bulky masonry as in foundations.

Most of the remainder of Art. 50 may be used without alteration. In general, specifications are not so strict for mill buildings as for bridges. This is to be expected, since there is less vibration, and stresses approaching the maximum are quite infrequent. We will call attention to the following changes:

In (d_2) least allowable thickness becomes $\frac{5}{16}''$.

(d_5) which penalizes angles connected by one leg may be omitted.

In (d_9), maximum slenderness ratios become 150 and 125 for bracing and other members respectively.

In (f_2), material, instead of being sub-punched and reamed, is punched $\frac{1}{16}''$ larger than diameter of rivet.

In (f_7) and (f_9), clauses forbidding welded and adjustable members are not enforced.

Complete specifications will cover details of floors, roofing, siding, windows, skylights, doors, ventilators, louvres, and so forth. To cover in sufficient detail the various kinds which might be specified would consume more space than is at our disposal.

Prob. 131. What weight will be saved in each roof truss

of 80 feet span by substituting medium for soft steel? Length of bay 25 feet, load 60 lbs. per sq.ft.

Art. 132. Computations for a Steel Frame Mill Building

Let it be required to design a machine shop with a space not less than 200×50×15 feet high available for a 15-ton crane. Underneath it is to be clear of columns except for the support of a mezzanine floor, 10 feet high, about 20 feet wide, extending the whole length on one side, to be used for light machinery. Owner will build first floor himself. Locality, Wheeling, W. Va.

Building will be about 220 feet long and of a section seen in Fig. 132*a*. In the main we shall use specifications of preceding article. For the constant *C* in the formula for the wind pressure,

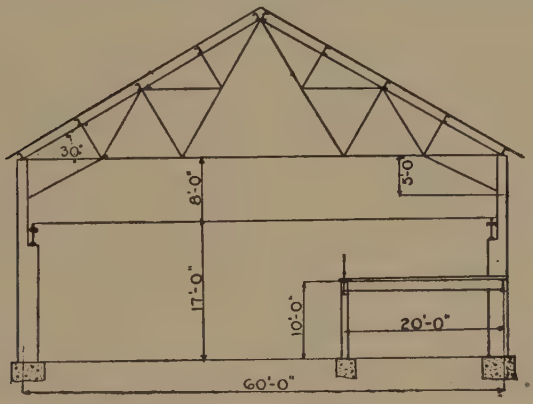


FIG. 132*a*.—Section of the Building.

we will allow 25. This will make pressure against sides 25 and against roof 20 lbs. per sq.ft. Consider snow to weigh 12 lbs. per horizontal sq.ft.

With $n=10$ in equation (4) of Art. 123, we find economical length of bay to be about 16 feet. The slate roof will be supported by 4″ hemlock plank amply safe to sustain the loads. The weight of the two will be estimated at 17 lbs. per sq.ft. The total on the purlin will be about 55 lbs. per sq.ft. or 480 lbs. per lin.ft., which we will consider as perpendicular thereto. Material is to be soft steel.

1 [10″ @ 20# has allowable span of 17.5′; use 13 panels @ 16′ 11″
1 [12″ @ 25# " " 21.7′; use 10 " @ 22′ 0″
1 [15″ @ 33# " " 28.6′; use 8 " @ 27′ 6″

We will divide into 10 panels, although 13 would probably be more economical in first cost.

The siding will be composed of $1\frac{3}{4}$″ matched sheathing supported by girts spaced five feet apart. To carry the horizontal wind load of 25 lbs. per sq.ft., we will use [′s 7″ @ 12.25#. One of these placed upright is assumed to carry the vertical load.

Crane Girders

The crane girders must be figured for two concentrated loads of 37,000 lbs. each,* 12 feet apart. This will cause a maximum live moment of 2,580,000 in. lbs. A 24″ I @ 80# would be a little weak and would lack transverse strength for top flange, so we add one [15″ @ 33.0# giving section seen in Fig.

Fig. 132*b*.
Section of Crane Girder.

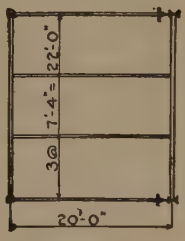


Fig. 132*c*.
Arrangement of Mezzanine Floor.

132*b*. *I/c* is 196, making tension for live load 13,200 lbs. per sq.in. That for dead load estimated at 130# per lin.ft. is 500 lbs. per sq.in. *I/c* in compression is 340, making live and dead vertical stresses 7600 and 300 lbs. per sq.in., respectively. Transversely, *I/c* for the top flange, including one-half of the I-beam, is 44.5, making unit stress for 10% of the vertical load, 5800. Total compression, 13,700 lbs. per sq.in.

* Includes impact. Span considered to be 57 feet.

Mezzanine Floor

This will be figured for 150 lbs. per sq.ft. live. Panel lengths will be the same as for the roof. $2\frac{3}{4}''$ yellow pine plank will weigh 10# per sq.ft. and at 1500 lbs. per sq.in. has an allowable span of 9′ 8″. To increase capacity of floor for local loads, and to lighten floor beam, we will divide bay into three panels as seen in Fig. 132*c*. The load on the stringers is 1200 lbs. per lin.ft., and a 15″ I @ 42# will be specified. For floor beams we have two concentrations of 12,000 lbs. each, causing a moment of 1,056,000 in.lbs. and requiring one I, 18″ @ 55#.* The load on column will be 38,000 lbs. An 8″ Bethlehem H @ 31.8# is ample.

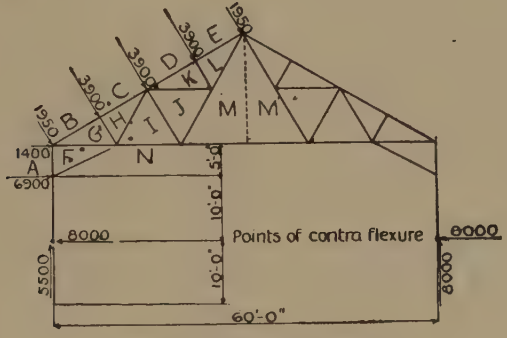


Fig. 132*d*.—Wind Loads.

Loads for Truss

From formula (2), Art. 123, using $w=55$, $a=22$, $S=14{,}000$, and $L=60$, we find the dead load of the roof truss to be 5300 lbs. The total dead panel load is $\frac{5300}{8}+17\cdot8.7\cdot22+25\cdot22=$ 4470, say 4500 lbs. For snow, $12\cdot7.5\cdot22=1980$, say 2000 lbs. For wind, $20\cdot22\cdot8.7=3830$, say 3900 lbs. Fig. 132*d* shows loads and reactions for this case. Horizontal components were assumed to be evenly divided between the reactions, with posts fixed at both ends.

* There is sufficient margin to care for the dead load of floor beam, which is not considered in the above.

Design of Roof Truss

Material Soft Steel. $\frac{1}{2}''$ Connection Plates. Rivets $\frac{3}{4}''$ dia. Values below are in kips * and inches.

Member.	Dead Stress.	Snow Stress.	Wind Stress.		Total.		Use.	Length.	Rad. of Gyr.	Area.		Allowable.	
			L.	*R.*	*T.*	*C.*				Gross.	Net.	Unit.	Total.
BG	C 31.6	C 14.0	C 24.6	T 8.0	....	70.2	2 L's $5\times3\frac{1}{2}\times\frac{7}{16}''$	104	1.53	7.06	....	C 10.3	C 73.0
CH	C 29.4	C 13.1	C 24.6	T 8.0	....	67.1							
DK	C 27.2	C 12.1	C 16.2	C 4.1	....	55.5							
EL	C 25.0	C 11.1	C 16.2	C 4.1	....	52.3							
FG	T 27.4	T 12.2	T 3.8	T 9.0	48.6	....	2 L's $3\times3\times\frac{3}{8}''$	...	....	4.22	3.57	T 14.0	T 50.0
IN	T 23.5	T 10.4	T 11.9	C 8.0	45.8	....							
MO	T 15.7	T 7.0	C 1.1	C 1.1	22.7	....	2 L's $3\times3\times\frac{5}{16}''$	...	....	3.56	3.02	T 14.0	T 42.3
FN	0	0	T 19.1	C 26.8	19.1	26.8	2 L's, $4\times3\times\frac{5}{16}''$	134	1.27	4.18	3.64	T 14.0 C 8.1	T 51.0 C 33.8
GH	C 3.9	C 1.7	C 3.9	....	....	9.5	2 L's, $2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$	60	0.76	2.94	....	C 9.6	C 28.2
KL	C 3.9	C 1.7	C 3.9	....	....	9.5							
HI	T 3.9	T 1.7	T 14.0	C 14.0	19.6	10.1	2 L's $3\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$	120	1.11	3.56	3.02	T 14.0 C 8.0	T 42.3 C 28.5
JK	T 3.9	T 1.7	T 3.9	....	9.5	....	2 L's, $2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$	...	....	2.94	2.40	T 14.0	T 33.6
IJ	C 7.8	C 3.5	C 12.9	T 7.0	....	24.2	2 L's, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$	120	1.11	3.56	....	C 8.0	C 28.5
JM	T 7.8	T 3.5	T 12.9	C 7.0	24.2	....	2 L's, $2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$	...	....	2.94	2.40	T 14.0	T 33.6
LM	T 11.7	T 5.2	T 16.8	C 7.0	33.7	....							
MM'	....	....	....	....	....	....	1 L, $2\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$	...	....	1.47	1.20	T 14.0	T 16.8

Columns

For design, we will divide column into two parts; that above the crane, 10 feet long; that below, 15 feet long. Bracing in sides will support their junction. Assume that each column carries tractive force due to its own reaction and that longitudinal knee braces extend five feet below rail. Fig. 132*e* shows the section which we will use.

* 1 kip = 1000 lbs.

	Upper Part.	Lower Part.		
		Direct and Wind Loads.	Direct Wind and Transverse Crane.	Direct and Long Crane.*
Dead load in kips	18.0	32.0	32.0	32.0
Snow load in kips	8.0	8.0	8.0	8.0
Wind load in kips	14.1	8.0	8.0	...
Crane load in kips	....	54.0	54.0	54.0
Mezzanine floor in kips	....	38.0	38.0	38.0
Maximum	40.1	140.0	140.0	132.0
Bending moment in kip.in	960	960	2062†	389‡
Use	1 Pl. 14×$\frac{7}{16}$ 4 L's, 6″ ×4″×$\frac{7}{16}$″	1 Pl., 28″×$\frac{7}{16}$″ 6 L's, 6″×4″×$\frac{7}{16}$″ 1 [15″ @ 33#		
Area in sq.in	22.9	47.3	47.3	47.3
Radius of gyration in ins	2.49	3.32	3.32	3.32
Moment of resistance $=I/c$, in ins.	107	312	312	51.0§
Flexural stress in lbs. per sq.in	8970	3080	6600	7630
Direct stress in lbs. per sq.in	1750	2960	2960	2790
Total	10,720	6040	9560	10,420
Allowable	11,200	11,000	14,300	11,000

Column Base

The masonry will be most severely tested by wind and transverse crane loads. Under these circumstances allowable stress on Portland cement concrete is,

$$250 \cdot 1.30 \cdot 1.50 = 490 \text{ lbs. per sq.in.}$$

The load is 140,000 lbs., which includes column-siding and so forth. The moment is 2,060,000 in. lbs. Letting b represent the breadth and d the depth of the base, we have,

$$\frac{140{,}000}{bd} + \frac{6 \cdot 2{,}060{,}000}{bd^2} = 490.$$

* Including wind gives practically the same section.

† Horizontal load 5.4 kips applied to a cantilever 17-foot beam.

‡ Horizontal load 5.4 kips. Length of fixed-ended post 12 feet.

§ For one channel and two angles.

A practical solution of which is, $b = 22.8''$ and $d = 40$. We will make $22''$.

For maximum tension, we have the same moment and a load of 94,000 lbs. Assuming that distance from center of gravity of compressive stresses to bolts is $29\frac{1}{3}''$, we have a tension due to moment of,

$$2{,}060{,}000/29.3 = 70{,}000 \text{ lbs.}$$

Subtracting therefrom one-half of the total compression leaves 23,000 lbs. Required net area becomes,

$$23{,}000/14{,}000 \cdot 1.30 = 1.26 \text{ sq.in.}$$

Use two bolts, $1\frac{1}{4}''$ dia. on each side, net area 1.77 sq.in.

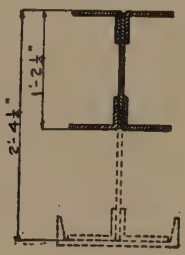


FIG. 132e.
Section of Column.

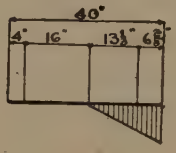


FIG. 132f.
Stresses in Base.

BRACING

We will design bracing to carry a shear equal to one-thirtieth safe compressive strength of members which it connects, when acting as a short strut. This force is supposed to be at right angles to the member. It will be noted that this gives lighter bracing for the bottom chord than for the top. While a little different from customary practice, Art. 127, we shall nevertheless use it. Insert bracing in second, fourth, seventh and ninth bays. The shear then equals area of one member braced times 10/4 times 14,000 divided by 30. The latter factor will be increased to 60 for the main columns to allow for the bracing by siding.

Member Braced.	Gross Area, Sq.in.	Shear, Lbs.	Secant.	Stress, Lbs.	Area, Sq.in.	Use.
Top chord.....	7.06	8,200	1.27	10,400	0.74	1″ O rods, upset.
Bottom chord...	4.22	5,000	1.36	6,800	0.49	1 L, $3\frac{1}{2}''\times2\frac{1}{2}''\times\frac{5}{16}''$ *
Columns.....	22.9	13,000	1.10	14,300	1.02	$1\frac{1}{8}''$ O rods, upset.

* Extra material inserted to give stiffness to reduce stresses due to own weight.

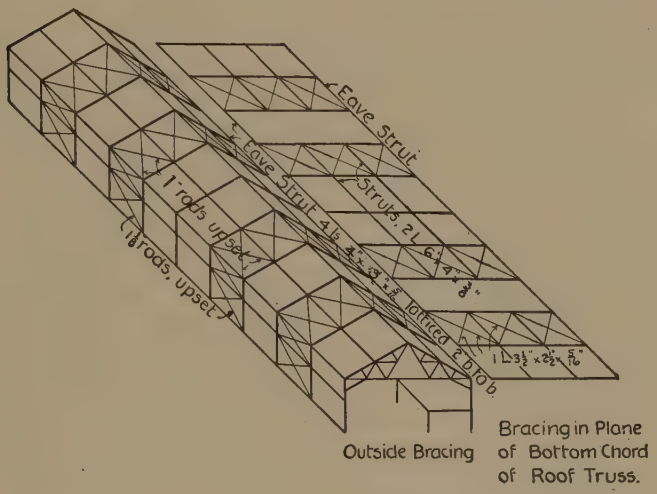


FIG. 132g.

The column under the crane will be efficiently knee-braced. For struts, the allowable slenderness ratio governs. One one-hundred and fiftieth of 22 feet is 1.76″. For top chord, use purlins reinforced by a $6''\times4''\times\frac{3}{8}''$ L, they are more or less stiffened by sag rods and roofing; for bottom chord, 2 L's, $6''\times4''\times\frac{3}{8}''$ with 6″ legs vertical and $\frac{5}{8}''$ apart; for eave strut, 4 L's, $4''\times3''\times\frac{5}{16}''$, with the 3″ legs latticed together. See Fig. 132*g*. A general drawing of important portions of the building will be found in Fig. 132*h*.

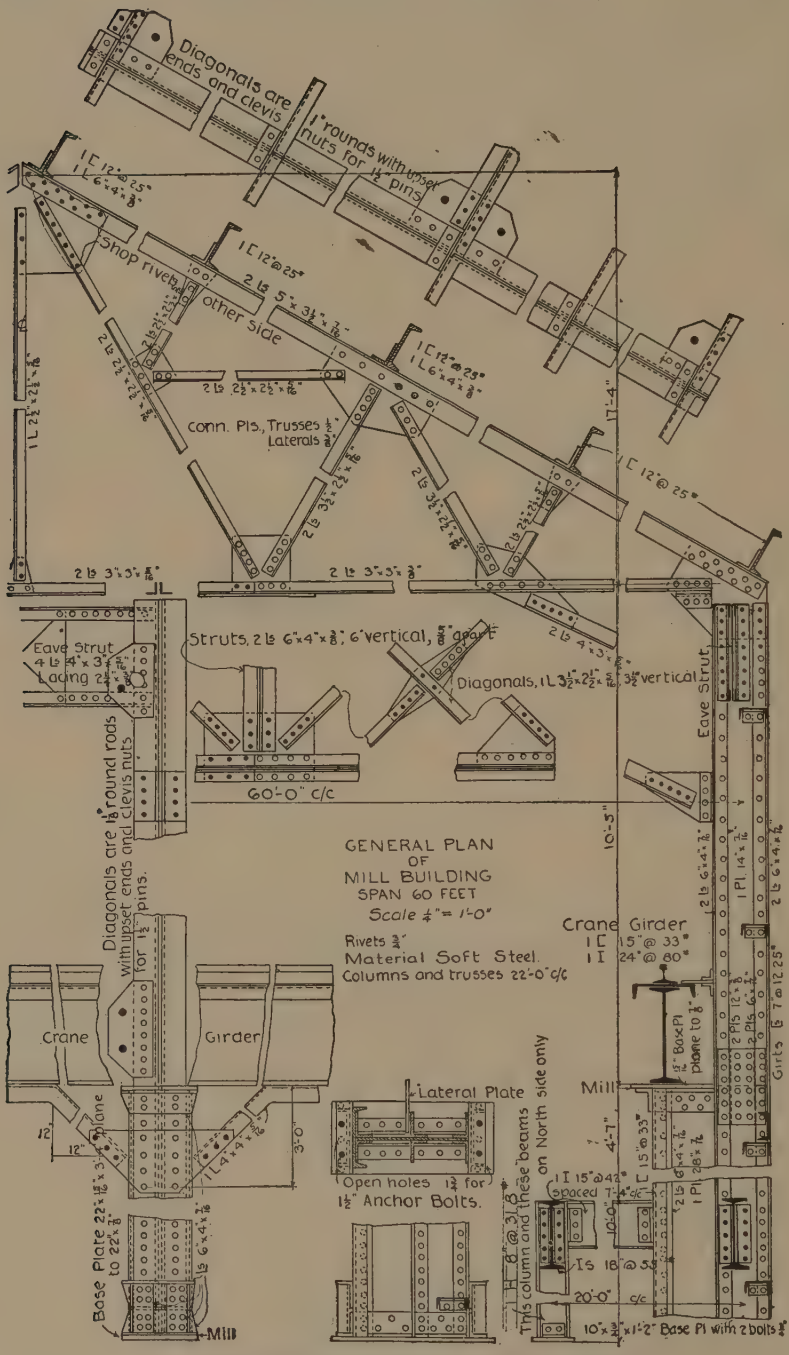

Diagonals are 1" rounds with upset ends and clevis nuts for 1½" pins
Shop rivets other side
conn. Pls., Trusses ½" Laterals ⅜"
Struts, 2 ⊔ 6"×4"×⅜", 6' vertical, ⅜" apart
Diagonals, 1 L 3½"×2½"×⅜" 3½' vertical
Eave Strut
Eave Strut, 4 ⊔ 4"×3"×⅜" Lacing 2½"×⅜"
60'-0" c/c
Diagonals are 1⅛" round rods with upset ends and clevis nuts for 1½" pins.
GENERAL PLAN OF MILL BUILDING SPAN 60 FEET
Scale ¼" = 1'-0"
Rivets ¾"
Material Soft Steel.
Columns and trusses 22'-0" c/c
Crane Girder
1 ⊏ 15" @ 33#
1 I 24" @ 80#
Crane
Girder
Lateral Plate
Open holes 1¾" for 1½" Anchor Bolts.
Base Plate 22"×1⅛"×3⅛" to 22"×⅜"
Mill
Mill
Girts ⊔ 7" @ 12.25#
1 I 15" @ 42# Spaced 7'-4"
1 Pl 26"×⅞"
⊏ 8" @ 31.8#
This column and these beams on North side only
⊏ 15 18" @ 58#
20'-0" c/c
10"×¾"×1'-2" Base Pl with 2 bolts ¾"

FIG. 132*h*.—General Drawing of Portions of Building.

REFERENCES

E. N., Engineering News; E. R., Engineering Record.

Foundries

Periodical.	Volume.	Page.	Type.	Main Span, Feet.	Length Bay, Feet.	Location.
E. R.	64	737	2	46	20	Dansville, N. Y.
E. R.	45	11	2	50	15	Harrison, N. J.
E. R.	45	59	1	7	20	Schenectady, N. Y.
E. R.	62	405	1	83	20	Pittsburgh, Pa.

Machine Shops

E. R.	68	234	2	30–35	25	Pittsburgh, Pa.
E. R.	49	148	2	60	18	Harrison, N. J.
E. R.	50	338, 394	2	66	30	Lynn, Mass.
E. R.	48	130	2	69	20	Schenectady, N. Y.
E. N.	41	226				
E. N.	45	300	1	70	...	Brooklyn, N. Y.
E. R.	52	179, 267, 414	2	76	25	Lynn, Mass.

Railroad Shops

E. N.	53	90	2	60	24	Sayre, Pa.
E. R.	51	77				
E. N.	43	339	..	69	20	Reading, Pa.
E. R.	62	134	2	70	22	Macon, Ga.
E. R.	57	50	2	74	22	Warwick, N. Y.
E. R.	57	97	2	78	26	Parsons, Kans.
E. R.	49	378	2	80	22	Montreal, Can.

STEEL MILLS

Periodical.	Vol.	Page.	Type.	Span.	Bay.	Location.
E. R.	44	61	1	58.	20	Middletown, Ohio
E. R.	64	301	1	69	27	Sparrows Point, Md.
E. R.	57	244	1	76	28	Steelton, Pa.
E. R.	60	396	...	84	...	Gary, Ind.
E. R.	68	164	1	140	25	Youngstown, Ohio
E. R.	61	496	..	Various		Gary, Ind.
E. R.	61	120, 637	1	Various		Pittsburgh, Pa.
E. R.	64	273 et seq.		Various		Middletown, Ohio

STRUCTURAL SHOPS

E. R.	55	527	2	56		
E. R.	65	462	2	62	20	Newark, N. J.
E. R.	51	40	..	..	..	Boston Bridge Works
E. R.	49	763, 801	..	..	..	Cambria, Johnstown, Pa.
E. R.	48	360 et seq.	..	..	..	Steelton, Pa.
E. N.	40	194	2	100	..	Pencoyd, Pa.
E. N.	68	238		Various		Gary, Ind.
E. R.	65	704				
E. R.	48	620 et seq.		Various		Ambridge, Pa.
E. R.	49	23 et seq.				

CEMENT PLANT

E. N.	51	243	2	Vari	ous	Bay City, Mich.

Prob. 132. Design above building for same loads and allowable stresses using 13 panels instead of 10.

CHAPTER XI

HIGH OFFICE BUILDINGS

Art. 133. General

In the last chapter, we considered the design of buildings for the handling and manufacture of bulky goods; conditions which demand a low structure and a location in the suburbs or the country. The executive quarters of the corporation, however, are often situated in the nearest large city, with branch offices elsewhere. Its quarters must be central in order to facilitate intercourse with those other concerns with which it does business. Hence we find in every large city a section where land is very valuable and high buildings are necessary to develop a fair return from the property.

Since the introduction of "cage construction," explained below, there has been a continuous drift towards higher buildings. The highest now in existence (1914) is the Woolworth Building, Fig. 133*a*, 55 stories or 775 feet high.* Aside from the economic advantages to the owner of the lot, the community as a whole is a gainer through the greater concentration of business affected. On the other hand, its disadvantages are,

(*a*) The exclusion of the sunlight from the streets and adjacent buildings.

(*b*) Congestion of traffic in the streets.

(*c*) Difficulty in providing suitable fire protection at such heights.

(*d*) Overcrowding sewer and water systems designed for a much smaller population.

(*e*) Appearance of skyscrapers of varying heights, side by side with less pretentious buildings.

There are three types:

* See Engineering Record, Vol. 63, p. 591; Vol. 65, pp. 177, 220; Vol. 66, p. 97; Vol. 68, p. 22.

FIG. 133*a*.—Woolworth Building, New York City.

(*1*) Exterior walls carry own weight and some of the adjacent floor load, the remainder being borne by the interior columns of steel or cast iron.

(*2*) Here beams and columns at the edge of the wall sustain the entire weight of the roof and floor, allowing the wall to support itself. It may be used for building up to seven stories in height.

(*3*) In this, the well-known cage construction, all loads including the weight of the wall, are carried at each floor level by the steel.

(*1*) is the common construction up to five stories. Above this limit, it will be found that the wall thickness which is demanded by the rules of Art. 142 becomes so great that a large portion of the space available for renting is absorbed: windows must be narrow in order to conserve the strength of the masonry, and, with the resulting thick walls, the light is poorly distributed. If the unit-bearing capacity of the ground under the footing be low, a large amount of masonry will be necessary to properly distribute the pressure over the soil; this is expensive and it takes much valuable space. The latter disadvantage may be overcome by using the grillage foundation, Art. 135.

Cage construction has three important advantages, which, in addition to the disadvantages of Types (*1*) and (*2*), practically eliminate weight-carrying walls from consideration in high buildings.

(*A*) Windows may be used for almost the entire exterior area without lessening the stability.

(*B*) The structure, if properly designed, is proof against hurricanes and earthquakes.

(*C*) It is necessary to fireproof these buildings, Art. 144; they are thus made safer against a conflagration than an ordinary structure.

The lot on which the building is erected is preferably square; nevertheless when it is not, it is customary to use the full area so far as it may be possible to get available office space and provide sufficient light. Buildings have been constructed in which no two sides were parallel or at right angles, but this is exceptional. Wherever the extra cost of the skew work is justified by the additional income, it should be done. The center line of columns

should be set far enough back to obtain the desired clearance from the building line; usually one or two feet. Building laws generally allow cornices, bay windows, and trim to encroach upon the street; on other sides, everything must clear unless a party wall, that is, a wall common to the two adjoining buildings, is arranged by the parties interested. Such an agreement of course precludes the possibility of light on that side. If, on the other hand, there are windows in the wall, the desirability of setting it back more than two feet should be investigated in view of the possibility of another building being erected on the adjoining lot.

Steel Cage Construction

Inside of these lines as just determined, the space is *subdivided* as indicated in *Art. 134*. *Bases and footings*, *Art. 135*, are designed for the building, care being taken that the pressure on the soil does not exceed safe limits. From these foundations, the *columns*, *Art. 136*, are carried vertically to the top of the building. Connecting with these shafts at each floor level are two sets of beams, *Art. 137*, at right angles to each other; *floor stringers* and the *floor girders* carrying the intermediate floor stringers, Fig. 133*b*. As their name implies, these stringers support the *floor*, *Art. 138*. At the top, trusses or a similar set of beams sustain the *roof* of the structure, *Art. 139*. The *exterior wall*, *Art. 142*, rests upon the outside beams and stringers. Its functions are to carry its own weight, retain the heat in the building, admit light and keep out the weather. Between the columns and in their plane, is placed the *wind bracing*, *Art. 141*, which stiffens the structure and prevents any distortion by the wind. Inside the building is divided by *partitions*, *Art. 143*, into *rooms* of suitable size, *Art. 134*. Almost every office building contains enough furniture, papers, and other combustible material to make a fire which would heat unprotected steel so that it would lose the greater part of its resisting power. The collapse which would follow would ruin the building and endanger many lives. To prevent this

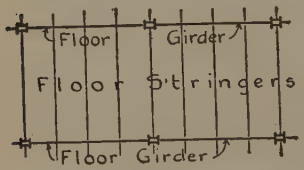


Fig. 133*b*.—Floor Framing.

and the spread of the fire, the structural materials of the building should be *incombustible* and non-conductors of heat, *Art. 144*. Also passages allowing the transmission of heat from room to room and floor to floor should be avoided as far as practicable. Enough can be done in this regard to render the tall building much safer than those of moderate height as usually designed. The many *conveniences* afforded by up-to-date construction are treated briefly in *Art. 145*.

Construction and Erection

It is usually necessary to tear down an existing building before beginning on the foundations. During this demolition, construction of foundations, and erection of new building, there is a large and increasing sum which is bringing no return; this includes the value of the site, improvements, and amount already spent on the new structure. To make this loss a minimum, detailed plans are prepared in advance and erection is urged with phenomenal speed. It is now possible to replace the old building by a new one, twenty-five stories high, in a year. Let us study the methods by which this is accomplished.

Derricks are the common means for hoisting the steel into place. They may be movable or fixed, guyed by wire ropes or fastened to towers of steel or of wood. Perhaps the fixed guyed derrick with mast 70 feet long and boom 10 feet longer, best represents the usual practice. The location and number should be such that the entire area of the building is commanded as well as the street where the material is delivered. The arrangement should, as far as possible, avoid rehandling and the moving about horizontally of fixed derricks. They may either erect all columns which are spliced on the floor it is on and the floor above and then shift to the latter; or they may erect for two floors in advance. They are usually moved upward by separating boom and mast and using each in turn as a gin-pole to hoist the other. In this way, from two to four stories per week may be erected.

Great care should be taken to keep the columns vertical. If not plumb, they may be forced over by fastening top to other columns by two or more adjustable wire ropes. If the milled ends of the columns do not bear true, shims, thin wedge-shaped

pieces of steel, may be inserted; with good workmanship, however, they will be unnecessary.

For inferior work, connections are sometimes bolted permanently. But the usual practice is to temporarily bolt up one-third of the open holes, the rivet gang following about three floors below, filling all holes with rivets driven by hand pneumatic hammers. These rivets will cost about 8 cents apiece, while erection will average about 35 cents per hundred pounds. It will rarely be found that erection stresses occur which test the structure more severely than the final loading. About the only danger is from the storage of building material.

As soon as the steel work of a floor has been riveted, floor and its fireproofing are laid, and the exterior walls are built up, setting in the window frames as they are reached. One advantage of the steel cage construction is that if the material for some of the masonry be delayed, it may be omitted for the time being, as each wall and floor rests directly on the steel. Also work at different heights may be prosecuted simultaneously. Material is usually hoisted to the different floors by a rough elevator centrally located. After the completion of this masonry, the wooden floor sleepers are laid and fastened to the beams. The electricians and plumbers next install their "roughing."* Then the cinder fill is placed, columns are fireproofed, and partitions erected. The building is now plastered and finally windows, doors, stairways, trim, and so forth are put in position.

Cost

Let A represent the average area of each floor in sq.ft.
Let N represent the number of floors, then,

Weight in lbs. of steel in floor is $15\ AN$.
Weight in lbs. of steel in columns is $0.7\ AN^2$.

Its cost per pound erected may be reckoned at base price of steel at Pittsburgh † plus freight from Pittsburgh plus 1 cent

* Conduits, ducts, mains, and pipes for mechanical conveniences.

† The price of ordinary rolled material in large lots. This is now (April, 1914), $1.20 per hundred pounds. The preeminence of Pittsburgh in this industry is such that when manufactured elsewhere, value is determined largely by the price at which Pittsburgh can deliver.

for beams, and plus 1.8 cents for columns. 40 cents per cu.ft. may be taken as a rough average figure for cost of entire building. 12 to 14 feet is usual height of story, but this may be increased for ground floor.

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Engineering Magazine, Vol. 13, p. 551.
Engineering News, Vol. 26, p. 534 et seq.; Vol. 27, p. 2 et seq.; Vol. 42, p. 325.
Engineering Record, Vol. 38, p. 11 et seq.
Engineers Society of Western Pennsylvania, Vol. 18, p. 217.
Journal of Franklin Institute, Vol. 141, pp. 47, 115.
Proceedings of Engineers Club of Philadelphia, Vol. 19, p. 1.

First Cost

Engineering News, Vol. 57, p. 160.
Engineering Record, Vol. 68, p. 21.

Operating Costs

Engineering Record, Vol. 48, p. 759; Vol. 50, p. 263; Vol. 68, p. 21.
Engineering and Contracting, Vol. 40, p. 125.

Erection

Engineering Record, Vol. 55, p. 696; Vol. 56, p. 530; Vol. 66, p. 256.
See also references for Art. 147.

Prob. 133. Find approximate cost of steel in building of Art. 147, when base is $1.30 per hundred.

Art. 134. Floor Plans

The division of the floor area of the building costs about 2% of the total, or say 1% of the entire investment. On the other hand, it affects vitally its renting qualities. For this reason, economy of partitions scarcely needs consideration, the sole idea being to procure the maximum rental. This will be accomplished when we obtain the largest available space consistent with,

(1) Good natural light.
(2) Convenient size and arrangement of rooms.
(3) Ease of access.
(4) Suitable toilet facilities.

The location of the columns is chosen partly to bring them within the partition walls, thus avoiding obstruction to the renting space. We might give them such a position that it reduced total cost of steel to a minimum, but dimes so gained would be lost in dollars of income. These partitions, Art. 143, are light affairs, carry no load from above, and add little to the floor on which they rest. Sometimes the floor plan does not show all partitions, the remainder being placed to suit the tenant. Also changes can readily be made. However, this should not be taken as an excuse for a poor design as alterations are much more expensive per unit than new work and they are limited in their scope by arrangements of windows, columns, elevators, plumbing, wiring, and so forth.

The offices should be placed near the natural light, preferably on a side facing a street, thus leaving the darker and less desirable portions for the elevators, corridors, and toilet rooms.

For size, number, and description of elevators, see Art. 145. They are usually placed together in a central location. Let us call the distance from entrance to elevator on ground floor plus distance to any given room from the elevator on the same floor, the *virtual distance*. Now that arrangement which makes the sum of the virtual distances a minimum is best. It is also desirable to keep the maximum virtual distance as low as possible.

Three or four feet gives sufficient width for stairways between floors. Two rises plus one tread, Fig. 134*a*, should be about

24″, the usual rise being 7 to $7\frac{1}{2}$″. Where stairs turn, a landing is to be inserted as shown in Fig. 134*b*. The designs shown in Fig. 134*c* are objectionable.

The uses of stairs are,

(*1*) For one or two floor trips.

(*2*) In case of insufficiency or breakdown of elevator accommodations.

(*3*) For fire or earthquake cutting off that section containing elevator.

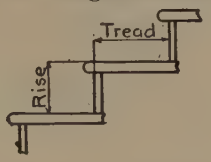


Fig. 134*a*.—Rise and Tread in Stairs.

As the use of the stairs is not economically important, they should be put where they will least interfere with the other parts of the building. They are often placed near or around the elevators, but this is objectionable on account of (*3*). A few details of construction will be given in Art. 137.

The corridors, which are the passages from the elevators to the rooms, should be four to five feet for side corridors and six to eight feet and even more in front of the elevators. Floor space should not be needlessly wasted in the halls; thus let Fig. 134*d* represent a typical floor of a 20-story building; to widen the corridors one foot would decrease the renting area 4000 sq.ft., representing when capitalized at 4%, a value of $200,000.*

Toilet rooms in detail will be treated in Art. 145. Separate accommodations for each sex must be considered. One large room for each may be furnished, sufficient for entire building,

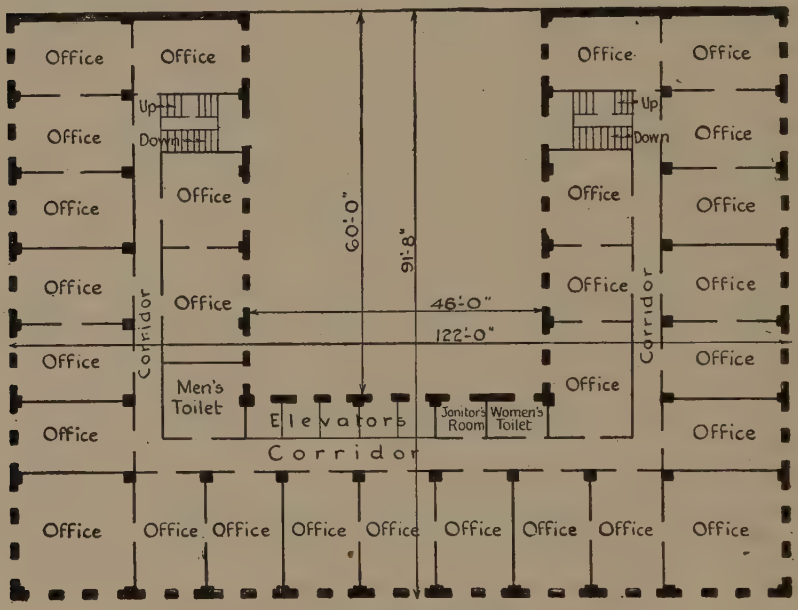
Fig. 134*b*.—Correct. Fig. 134*c*.—Objectionable.

Stairs with Turns.

or facilities provided on each floor. The number of toilets necessary may be obtained by adding two to one-fiftieth the number of permanent occupants which rely upon the room.

* Annual rental assumed at $2.00 per sq.ft.

In the absence of more definite information, allow one woman and four men to each office or every 250 square feet if not subdivided. For men's rooms, one-third should be urinals. The use of the above formula will show considerable economy for the single toilet rooms. But do not forget that it is much less convenient for tenants and will call for increased elevator accommodations. Compromise schemes such as, two general


Office
Office
Office

are objectionable, the loads which they would otherwise carry may be borne by plate girders or trusses. For single stores, 20 feet or thereabouts makes a nice front, and the column spacing may readily be made this amount, thus placing them in the partitions. Corridors are usually unnecessary, as the stores must have street frontage to command ground floor prices.

Above the first floor, the rooms are not less than 10 feet wide nor less than 12 deep. From this up to 16 by 24 is the usual size. Generally but one wall is exposed to the light. Of this, 40% should be windows if facing the street, or 50% for a light court, varying somewhat with the exposure and the size of the court. A rectangular arrangement of the rooms is cheaper and should be employed unless it makes too much of a reduction in the rental roll.

Prob. 134. With money at 5%, what sum could one afford to spend to reduce size of interior columns of Fig. 134*d* from 20″ square to 18″?

REFERENCE

Engineering and Contracting, Vol. 40, p. 516.

Art. 135. Foundations

The site of the future building is first excavated to the level of the bottom of the basement floor, which may be between 15 and 45 feet below the curb level. If there be more than 5000 cu.yds., it will justify the use of the steam shovel. To prevent disturbance of neighboring property, it will usually be necessary to line this excavation with tongued and grooved sheeting, 2 to 6″ thick, driven vertically and held by wales.* The latter may be braced from the other side or from a stake driven in the ground. The basement usually extends under the sidewalk; in order not to interfere while the vault† is being excavated and its wall and roof built, a platform 3 to 6 feet above curb level is built. It should be designed for 200

* Horizontal stringers supporting the plank sheeting.

† Space underneath the sidewalk.

lbs. per sq.ft. Ten to fifteen feet above it is placed a platform computed for 500 lbs. per sq.ft., on which may be unloaded materials for the building. It also serves to protect pedestrians from falling objects. A light roof is usually built between the two floors to keep off the dust.

Building laws often require that neighboring foundations whose stability may be threatened by the increased depth demanded by the high office buildings, shall be made secure at the cost of the owner of the latter. Sometimes this may be accomplished by shoring, that is, bracing as shown in Fig. 135*a*. After the masonry and walls are in place, these timbers may be removed and the hole at *a* bricked up.

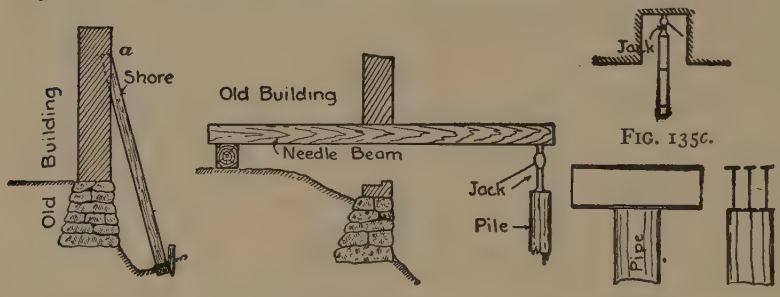


FIG. 135*a*. FIG. 135*b*. FIG. 135*c*. FIG. 135*d*.

Methods of Supporting Undermined Walls.

In unstable soil, however, the foundations of the neighboring buildings must often be renewed. The wall is then supported by *needle beams* as shown in Fig. 135*b* while its footings are underpinned, i.e., built up from a deeper and more secure base.

Another method is to tear out a vertical slot in the wall, and to force down an empty pipe by means of a jack, braced against the top of the slot, Fig. 135*c*. Sections of pipe are screwed onto that already driven until a stable stratum is reached. The inside of the pipe is then filled with concrete and beams placed over the top as shown in Fig. 135*d*.

The ground on which the structure is to rest may be (*a*) rock, (*b*) firm soil, (*c*) unstable soil. Rock is leveled up, a few inches of concrete placed thereon, and we are ready for the base of the column, which should not bring upon the concrete, a unit stress exceeding that allowable by the specifications in

Art. 146. Permissible pressure on soils seldom exceeds 100 lbs. per sq.in., hence extensive footings are required for (*b*) and (*c*). Here there are two very important principles,

(*1*) *Design each footing so that the center of gravity of the applied load coincides with that of the resisting forces.* The latter will be at the center of gravity of the area of the bottom of the footing in the case of a uniform soil. Avoid, if possible, those which are not uniform; it is difficult to judge correctly of their relative strength. For the same reason, combined footings are objectionable since the center of gravity of the applied forces will vary with the position of the live load on the tributary area. In this case, the center of gravity of the dead plus the probable live loads should be used.

(*2*) *Design footings to make the settlement uniform.* If the soil be homogeneous, this will be accomplished by making the unit pressure due to dead load plus probable live, the same on each footing. The maximum pressure due to all loads must not exceed prescribed values. It is impossible to prevent settlement unless foundations are on solid rock; however, several inches subsidence will do no harm if it be equal at all points. Hence the desirability of avoiding a foundation, a portion of which is on rock while the other is on a compressible soil, Fig. 135*e*. If impracticable to avoid by going deeper and stepping the rock, make footings on the latter small and those on soil large.

The following are the principal types of foundation,

(*a*) *Continuous.*
(*b*) *Masonry Footing.*
(*c*) *Grillage or Raft Footing.*
(*d*) *Pile.*
(*e*) *Caisson.*

(*a*) *Continuous*

Here a heavy mass of masonry, usually of concrete, sometimes reinforced, extends over the area of the building. However, centers of gravity of applied loads and footing area are not likely to coincide, and a considerable thickness of masonry is required to properly distribute pressure. For these two reasons, this method cannot be recommended.

(*b*) *Masonry Footing*

Formerly piers were often made of stone or concrete masonry as shown in Fig. 135*f*. Considering pressure as uniformly distributed, on each layer, they are beams, loaded as shown in Fig. 135*g*.

FIG. 135*e*.—Improper Foundation.

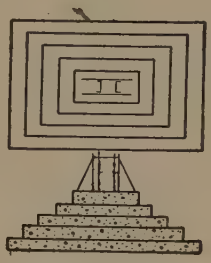


FIG. 135*f*.—Isolated Masonry Footing.

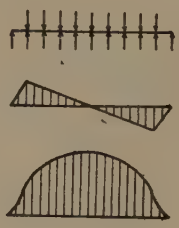


FIG. 135*g*.—Loads, Shears, and Moments for One Layer of Footing.

Shearing and flexural stresses must not exceed safe limits for concrete or stone. In the latter, the projection should be not more than half of a single stone.

Its use nowadays is confined to small footings. Allowable offset in plain masonry is limited, much room is required that would otherwise produce revenue, and it is expensive.

FIG. 135*h*.—Masonry Wall Foundation.

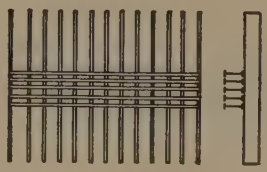


FIG. 135*i*. Isolated.

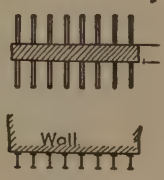


FIG. 135*j*. Wall.

Grillage Foundations.

Foundations for a wall are often built similarly, a section at any point being as shown in Fig. 135*h*. As in the other forms of this type, it is too expensive for anything but small footings.

(*c*) *Grillage or Raft Footing*

Allowable offset may be very much increased by using steel beams incased in concrete. A grillage foundation for a single load is shown in Fig. 135*i*; for a wall, in Fig. 135*j*.

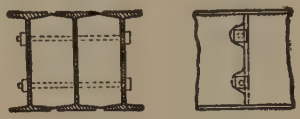


FIG. 135*k*.
Separators for an I-beam Grillage.

Reinforced concrete may also be employed. While an excellent and economical construction, it will not be treated here. In the I-beam foundation, the steel is supposed to carry the entire load, although the concrete doubtless assists. The method of computation is similar to that already given for type (*b*). Beams must be investigated for both shear and moment. The deeper I-beams are the more economical, 15 to 24″ being common depths. The lower limit is preferable for the bottom layer because it gives a better distribution on the foundation. Underneath this bottom layer is placed 6 to 12 inches of concrete, while at least 3″ protects the steel elsewhere. The beams must be spaced not farther apart than their depth. Sufficient room between the flanges should be allowed for placing the concrete, this distance being at least 2″. To maintain spacing, gas pipe or ordinary separators, Fig. 135*k*, are employed. The former consist of bolts passed through an ordinary gas pipe. A space of about an inch is left between grillage and base of columns, also between successive tiers of grillages. These spaces are grouted up while upper part is temporarily carried by wedges. For extremely heavy loads, built-up beams may be used; when shear alone is large, two plates may be riveted onto the I-beam or a plate fastened between two channels.

(*d*) *Pile*

Often material at depth where it is desired to place footings is untrustworthy. To go deeper in such a situation, especially in the presence of water, would endanger neighboring streets and buildings. A solid foundation may be secured by driving round timbers about one foot in diameter, called "piles," of sufficient length to reach a firmer stratum. These will carry safely not more than 50,000 pounds, this amount varying with

the soil. Wooden piles should be placed only where they will always be wet; otherwise they will rot. (Art. 3.) This objection does not apply to concrete piles, which are now extensively used. They will bear as much as 100,000 pounds, depending likewise upon the soil. The usual spacing of piles is 20 inches to 3 feet, center to center. Their arrangement should be such as to bring the center of gravity of the piles under the center of the column. One or two feet of earth is removed from around the pile, concrete is placed in the excavation and built up on top of it, the angle of inclination of the masonry with the vertical being about 30 degrees, Fig. 135*l*. Grillages may also be used to transfer the load from the column to the piles. If of wood, they too must be kept below the water line. In this connection, it is well to call attention to the danger arising from a lowered level of the ground water caused by driven wells, sewers, tunnels, or drainage into some leaky cellar in the vicinity.

FIG. 135*l*.
Pile Foundation.

(*e*) *Caisson*

This type is essentially a large single pile, either round or square, made of brick or concrete masonry. Like pile foundations, they are used where soil at footing is too unreliable for proposed loading. They have the following advantages over ordinary pile foundations:

(*1*) They allow a larger pressure on the bottom of the footing, since they may be driven with but a bare clearance.

(*2*) A large column will carry more per unit area than a small one.

(*3*) Perfect contact with bearing area is assured.

(*4*) Possibility of injury beyond reach of inspection or repair is avoided.

(*5*) Less danger to adjoining buildings. The ground where wooden piles are driven must be excavated to the water line.

We find then that, in spite of their increased cost, caissons are growing in favor in consequence of their safety and rigidity. There are two methods of sinking them.

In the hydraulic method, the caisson is a pipe, usually of steel, open at each end. It is sunk by jets around the lower edge and by weights placed on top. The dirt is excavated after bottom is reached and then masonry is built up inside. On top of the masonry, a grillage is laid to receive the column base. But this method can be used only where the soil at the bottom has sufficient stability to resist the inrush of water as the caisson is pumped out. Otherwise the more expensive pneumatic caisson must be employed.

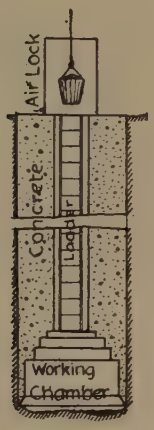


Fig. 135*m*.
Pneumatic Caisson.

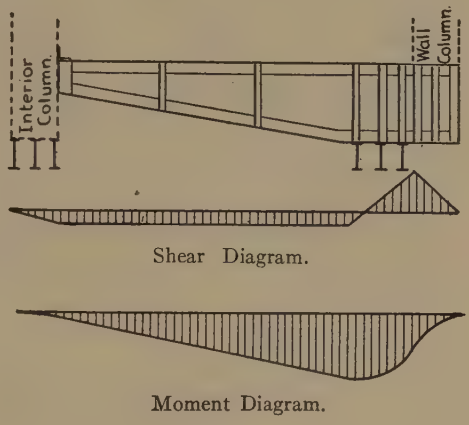


Fig. 135*n*.
Cantilever Girder.

In this method, a transverse bulkhead about 8 feet from the bottom of the pipe forms a working chamber, Fig. 135*m*. On top of this bulkhead and inside of the pipe is laid the masonry, space being left for an opening to the chamber below. Here is placed a double valve, forming an air lock which allows the passage of workmen and material. The caisson is forced down by the weight of the pier above, aided by the excavation of the material in the working chamber. Air pressure is maintained therein to exclude water and silt. After the rock is laid bare, it is leveled off and the working chamber with its shaft filled with masonry. The grillage for the column is similar to that for the hydraulic caisson.

Cantilever Girders

As previously indicated, center of gravity of loads and corresponding foundations must coincide. But the columns for the exterior walls are of necessity quite near the property line. To avoid trespassing, we have recourse to the cantilever girder. Figs. 135*n*, 135*o* and 135*p* illustrate typical examples. The pin bearing shown in Fig. 135*o* prevents concentration of pressure

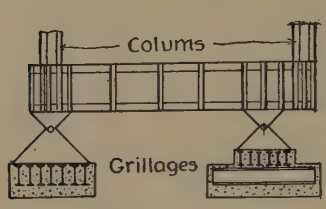


Fig. 135*o*.

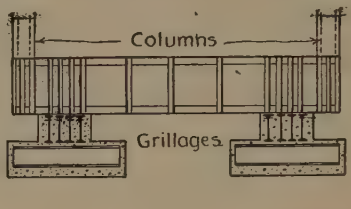
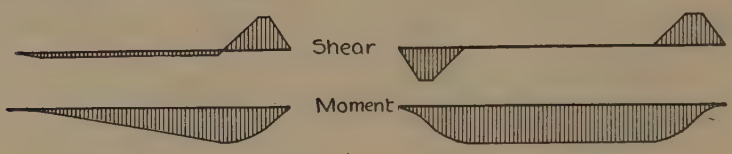


Fig. 135*p*.



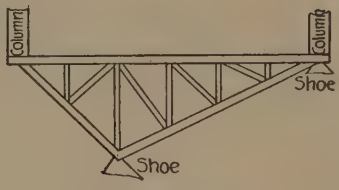


Fig. 135*q*.
Cantilever Girders.

on one edge of footing; but it is more expensive and little used. These girders are designed and built much like ordinary plate girders, Chapter VI. Unlike them, however, shear is generally more important than moment. Hence those parts in a plate girder which carry shear need special attention. The web is often $\frac{3}{4}''$ thick or has plates riveted on each side for this reason; stiffeners are placed close together, those under the columns and over the footings being figured for their loads, See Art. 147 for an example. Sometimes this girder is replaced by a riveted truss as shown in Fig. 135*q*.

Cantilever girders occasionally have more than two supports. This gives the well-known advantages and disadvantages of continuous beams, which American engineers seek to avoid.

Retaining Walls

To hold back the earth, three types of retaining walls are employed, Figs. 135*r*, 135*s*, and 135*t*. In the former, the top width is usually made about two feet and the bottom breadth 0.4 to 0.7 of the height, varying with the backfilling and the foundations. In Fig. 135*s*, the top width is about 12″ while the base will be about the same as for Fig. 135*r*. For full design,

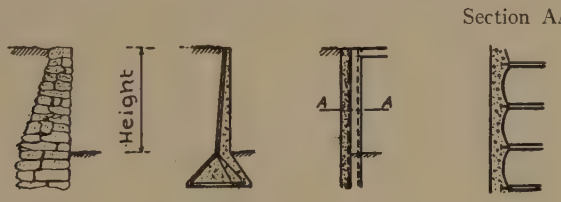


Fig. 135*r*. Fig. 135*s*. Fig. 135*t*. Fig. 135*u*.

Retaining Walls.

Fig. 135*v*.—Load, Shear, Moments.

see works on reinforced concrete. In Fig. 135*t*, the horizontal beam at the top also acts as a strut for the vertical beam. The pressure of the earth may be estimated as equal to that of a fluid weighing from 10 to 20 pounds per cubic foot. The load, shear and moment diagrams for this case are shown in Fig. 135*v*. The I-beams are figured for the stresses, the concrete serving to protect them and acting either as a beam or arch to carry the loads to the I's. Beams are commonly spaced four to six feet on centers, and concrete is one to two feet thick.

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See also references for Arts. 133 and 147.

Prob. 135. Design column footings like Figs. 135*f* and 135*i* for a load of 800,000 pounds and compare costs. Pressure on foundations not to exceed 10,000 lbs. per sq.ft. Concrete 25 cents per cu.ft. and steel 3 cents per lb. Allowable stresses in lbs. per sq.in.:

Steel, 16,000 flexure; 12,000 shear.
Concrete, 50 flexure; 40 shear.

Art. 136. Columns

Referring to Art. 56, considerations (*1*), (*2*), (*3*), (*4*), and (*5*) are important. (*6*) is often neglected and (*7*) does not apply. Capacity for increase of section with efficient and economical connections is also very desirable. In order to keep beams in a vertical tier alike, it is better to make outside dimensions of column the same throughout its length; or if this cannot be done, to change it as seldom as possible.

The "box" section, Fig. 56*g*, either with or without channels, has been the favorite. The built I, Figs. 56*f* and 56*g*, and the

Z bar, Fig. 56*r*, are also used a great deal and other types are occasionally seen. In recent years, the built I has become popular, and it is probably best except for the very highest buildings. The columns are usually fabricated in two-story lengths, sometimes, however, in three-story lengths. Splices are placed $1\frac{1}{2}$ to 2 feet above the floor and they are arranged to stagger as shown in Fig. 136*a*. Rarely are they designed to occur together at each alternate floor.

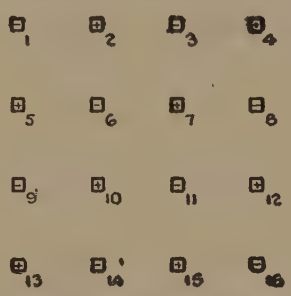


FIG. 136*a*.—Arrangement of Splices.
—Splice at 1st, 3rd, 5th, etc., floor.
+Splice at 2d, 4th, 6th, etc., floor.

There are two ways of making this splice.

(1) *Cap Plate.*
(2) *Continuous.*

For (1), each end of the column is milled to length and angles are set flush on two or four sides and a cap plate is riveted thereto, Fig. 136*b*. It is commonly bolted to cap of adjoining column. However, this type is no longer in favor for high office buildings due to its lack of stiffness. This may be remedied by substituting (2), the continuous splice. Here plates are shop

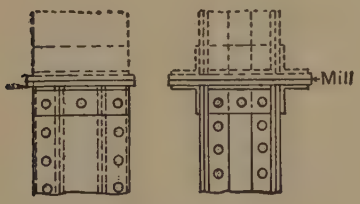


FIG. 136*b*.
Cap Plate Splice.

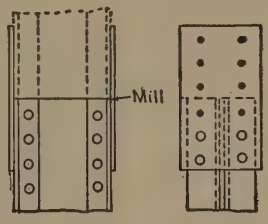


FIG. 136*c*.
Continuous Splice.

riveted to the lower column and field riveted to the upper, the section being first milled to ensure full bearing, Fig. 136*c*. Size of plates and number of rivets are determined by making bending strength of the splice equal to that of the full column.

Base

The size of the base plate for the shoe is determined by dividing load by allowable unit stress on top of foundation. The shoe may be of cast iron, cast steel, or structural steel. Fig. 136*d* shows a cast shoe which might be either of steel or iron. Fig. 136*e* gives a structural steel shoe. As stated in Art. 61, care must be taken that there is sufficient strength to properly distribute the load.

Column Schedules

A column schedule is a tabular statement of the shapes used in different columns on the different floors. That below is

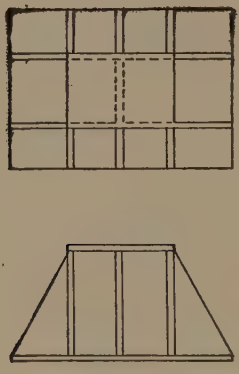


Fig. 136*d*.
Cast Shoe.

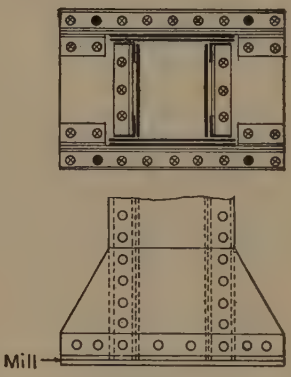


Fig. 136*e*.
Structural Shoe.

for the 7-story warehouse shown in plan in Fig. 136*a*. Successive changes in each type of column are illustrated by Fig. 136*f*. The dimensions at the top are constant throughout for that type. This makes the column splices effective and easily detailed and tends to keep the beams in a vertical tier alike.

The assumed dimensions for the building were 60 feet square with 16 feet between floors except first, which is 20 feet high. The loads were taken as follows in lbs. per sq.ft.: wall, 156, with full 16 foot-panel load at roof to allow for cornice; dead of floor, 125; dead roof, 60; live, floor, 200; live, roof, 40.

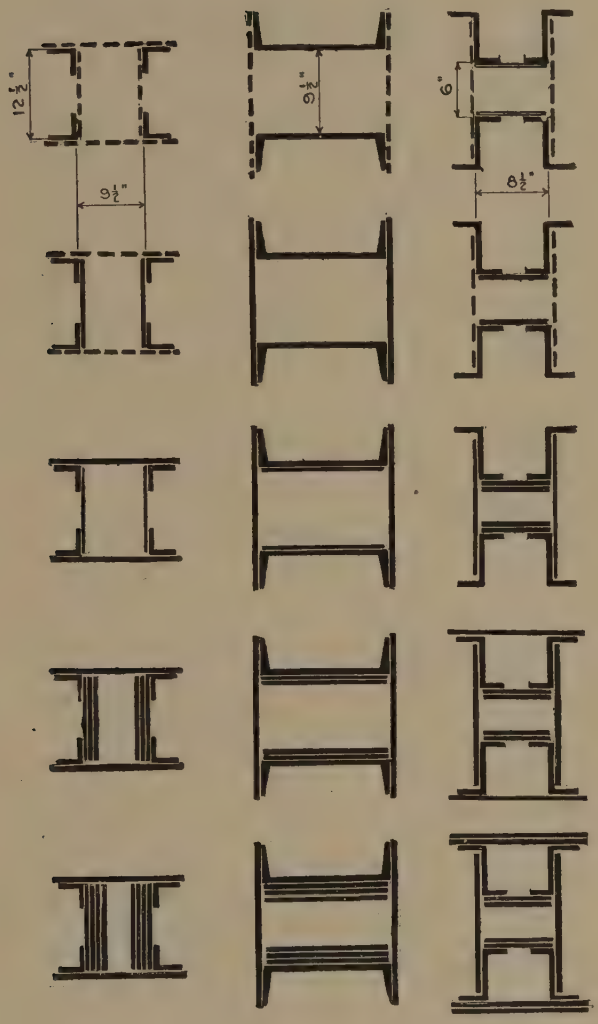


FIG. 136*f*.

Column #1 #3 #6

	Sub-Basement 16′–0″	Basement. 16′–0″	1st Floor. 20′–0″	2nd Floor. 16′–0″	3rd 16′–0″	4th 16′–0″	5th. 16′–0″	6th 16′–0″	7th 16′–0″
#1	$W=732$ 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 8 Pls. $12\times\frac{3}{8}$″ 2 Pls. $18\times\frac{3}{8}$″ $A=59.46$ $\rho=4.62$ $S=12.3$ $P=733$	650	555 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 4 Pls. $12\times\frac{3}{8}$″ 2 Pls. $12\times\frac{3}{8}$″, halfway 2 Pls. $18\times\frac{3}{8}$″ $A=50.46$ $\rho=4.80$ $S=11.5$ $P=580$	472	390 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 2 Pls. $12\times\frac{3}{8}$″ 2 Pls. $18\times\frac{3}{8}$″ $A=32.46$ $\rho=5.24$ $S=12.7$ $P=413$	308	225 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 2 Pls. $12\times\frac{3}{8}$″ $A=18.96$ $\rho=4.55$ $S=12.2$ $P=231$	143	60 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ $A=9.96$ $\rho=5.35$ $S=12.8$ $P=127$
#2	$W=1003$ 2 ['s 15″ @ 33 2 Pls. $18\times\frac{3}{8}$″ 4 Pls. $14\frac{1}{2}\times\frac{5}{8}$″ 2 Pls. $14\frac{1}{2}\times\frac{1}{2}$, $\frac{3}{4}$ way $A=84.04$ $\rho=4.57$ $S=12.2$ $P=1028$	888	760	645 2 ['s 15″ @ 33 2 Pls. $18\times\frac{3}{8}$″ 2 Pls. $14\frac{1}{2}\times\frac{3}{8}$″ 2 Pls. $14\frac{1}{2}\times\frac{3}{8}$, halfway $A=55.04$ $\rho=5.05$ $S=12.6$ $P=693$	530	415 2 ['s 15″ @ 33 2 Pls. $18\times\frac{3}{8}$″ $A=33.30$ $\rho=5.43$ $S=12.8$ $P=427$	300	185 $A=19.80$ $\rho=5.57$ $S=12.9$ $P=253$	70 2 ['s 15 @ 33
#3	$W=1003$ 2 ['s 15″ @ 33 2 Pls. $18\times\frac{3}{8}$″ 4 Pls. $14\frac{1}{2}\times\frac{5}{8}$″ 2 Pls. $14\frac{1}{2}\times\frac{1}{2}$″ $A=84.04$ $\rho=4.57$ $S=12.2$ $P=1028$	888	760 2 ['s 15″ @ 33 2 Pls. $18\times\frac{3}{8}$″ 2 Pls. $14\frac{1}{2}\times\frac{5}{8}$″ 2 Pls. $14\frac{1}{2}\times\frac{5}{8}$″, halfway $A=69.54$ $\rho=4.80$ $S=11.5$ $P=800$	645	530 2 ['s 15″ @ 33 2 Pls. $18\times\frac{3}{8}$″ 2 Pls. $14\frac{1}{2}\times\frac{3}{8}$″ $A=44.17$ $\rho=5.23$ $S=12.7$ $P=561$	415	300 2 ['s 15″ @ 33 2 Pls. $18\times\frac{3}{8}$″, halfway $A=33.30$ $\rho=5.43$ $S=12.8$ $P=427$	185	70 2 ['s 15″ @ 33 $A=19.80$ $\rho=5.57$ $S=12.9$ $P=253$
#4	$W=732$ 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 8 Pls. $12\times\frac{3}{8}$″ 2 Pls. $18\times\frac{3}{8}$″ $A=59.46$ $\rho=4.62$ $S=12.3$ $P=733$	650 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 6 Pls. $12\times\frac{3}{8}$″ 2 Pls. $12\times\frac{3}{8}$″, halfway 2 Pls. $18\times\frac{3}{8}$″ $A=59.46$ $\rho=4.62$ $S=12.3$ $P=733$	555	472 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 2 Pls. $12\times\frac{3}{8}$″ 2 Pls. $12\times\frac{3}{8}$″, halfway 2 Pls. $18\times\frac{3}{8}$″ $A=41.46$ $\rho=5.04$ $S=12.6$ $P=523$	390	308 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 2 Pls. $12\times\frac{3}{8}$″ 2 Pls. $18\times\frac{3}{8}$″, halfway $A=32.46$ $\rho=5.24$ $S=12.7$ $P=413$	225	143 4 L's $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$″ 2 Pls. $12\times\frac{3}{8}$″, halfway $A=18.96$ $\rho=4.55$ $S=12.2$ $P=231$	60
#5				Same as Number 2					

#6	$W=1080$ 950 4 Z's 6″ @ 29.3 4 Pls. 8 × $\frac{5}{8}$″ 4 Pls. 16 × $\frac{3}{8}$″ 2 Pls. 16 × $\frac{1}{2}$″ $A=94.52$ $\rho=4.00$ $S=11.7$ $P=1103$	820 690 4 Z's 6″ @ 29.3 4 Pls. 8 × $\frac{5}{8}$″ 4 Pls. 16 × $\frac{3}{8}$″ $A=78.52$ $\rho=3.86$ $S=10.4$ $P=817$	560 430 4 Z's 6″ @ 15.6 4 Pls. 8 × $\frac{9}{16}$″ 2 Pls. 16 × $\frac{3}{8}$″ $A=48.36$ $\rho=3.72$ $S=11.4$ $P=552$	300 170 4 Z's 6″ @ 15.6 2 Pls. 8 × $\frac{1}{2}$″, halfway $A=26.36$ $\rho=3.80$ $S=11.5$ $P=304$	40 4 Z's 6″ @ 15.6
#7	$W=1080$ 950 820 4 Z's 6″ @ 29.3 4 Pls. 8 × $\frac{5}{8}$″ 4 Pls. 16 × $\frac{3}{8}$″ 2 Pls. 16 × $\frac{1}{2}$, $\frac{3}{8}$ way $A=94.52$ $\rho=4.00$ $S=11.7$ $P=1103$	690 560 4 Z's 6″ @ 15.6 4 Pls. 8 × $\frac{9}{16}$″ 2 Pls. 16 × $\frac{3}{8}$″ 2 Pls. 16 × $\frac{3}{8}$″, halfway $A=60.36$ $\rho=3.92$ $S=11.6$ $P=700$	430 300 4 Z's 6″ @ 15.6 2 Pls. 8 × $\frac{1}{2}$″ 2 Pls. 16 × $\frac{3}{8}$″, halfway $A=38.36$ $\rho=4.00$ $S=11.7$ $P=450$	170 40 4 Z's 6″ @ 15.6 $A=18.36$ $\rho=4.30$ $S=12.0$ $P=220$	
#8		Same as Number 3			
#9		Same as Number 3			
#10			Same as Number 7		
#11		Same as Number 6			
#12			Same as Number 2		
#13			Same as Number 4		
#14		Same as Number 3			
#15			Same as Number 2		
#16		Same as Number 1			

FIG. 136g.—Farmers Bank Building, Pittsburgh, Pa. McClintic-Marshall Construction Co., Rankin, Pa.

In the schedule, W represents the total load; A, area; ρ minimum radius of gyration; S, allowable unit stress computed from the formula, $16000 - 90\frac{l}{\rho}$; and P, total allowable load; all in kips and inches. Where the section of a two-story length of column varies, quantities are given for the larger area.

For purposes of instruction, we have given three different types of columns for the same building. In practice this tends to prevent simplicity of details and should be avoided.

REFERENCES

Engineering Record, Vol. 58, p. 432. Bethlehem Sections.
Engineering News, Vol. 48, pp. 334, 337, 339, 544. Discussion.
See also references for Arts. 133 and 147.

Prob. 136. Using allowable stresses of Art. 146, compute column schedule above for built I sections with 4 L's $6'' \times 3\frac{1}{2}''$, $12\frac{1}{4}''$ back to back. Compute weight and compare with Bethlehem H sections.

Art. 137. Beams

Stringers

While the span of a floor stringer may vary from almost nothing to thirty feet, or even more in special cases, an average length is sixteen feet. For the customary loads, spans, and spacing in high office buildings, a rolled I-beam is sufficiently strong and most economical. As it readily fulfills all requirements, we find little else and that due to special circumstances. For example, channels give support much closer to the wall or they may be used to advantage in framing around an opening. Sometimes a long span floor construction is employed and stringers are omitted altogther.

Limiting Lengths

Let L = the length of the beam;
b = breadth of web of beam;
d = depth of beam;
c = distance to most strained fiber of beam;
I = the moment of inertia of the beam;
E = its modulus of elasticity;
S = allowable unit stress;
w = uniform load per lineal unit.

Beams carrying a plastered ceiling are likely to crack if the deflection, f, amounts to more than one-three-hundred-and-sixtieth of the span. For a uniform load per unit of length we have as the limit,

$$f = L/360 = 5wL^4/384\,EI. \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

From the flexural formula,

$$S = Mc/I = wL^2c/8I \text{ or } L^2 = 8IS/wc. \quad . \quad . \quad . \quad (2)$$

Substituting (2) in (1),

$$Ec = 37.5\,LS.$$

Substituting $d/2$ for c; 30,000,000 for E; and 16,000 for S, we obtain,

$$d = L/25.$$

The common rule is that one-twenty-fourth the length is the least allowable depth. If in the above, we omit the substitution for S, we have,

$$d = (L/25)\,(S/16{,}000).$$

In investigating the minimum length some engineers prefer to proceed in the manner indicated in Art. 86. However, beam is more or less supported by floor, fire proofing, and other beams. Even if we follow analysis referred to, shear is ordinarily more important. We will therefore investigate to find point where shear rather than moment governs. The algebraic solution consists in finding the length for which shearing and flexural stresses will reach allowable values under the same

load. Stress due to shear $=wL/2bd$. Stress due to moment $=wL^2c/8I$. Permissible stress for shear is usually taken as three-fourths that in flexure. Stating this as an equation, simplifying and substituting $\frac{1}{2}d$ for c,

$$L=32I/3bd^2.$$

The value of I/bd^2 for I-beams seldom exceeds 7 inches.* Hence beams less than 74 inches, or, let us say, 6 feet long, should be investigated for shear. This applies for a uniform load or concentrations applied somewhat equally along entire length. For a single load at the middle, it is on the safe side. For loads near the end, it is unsafe.

Tie Rods

Tie rods are round rods, threaded for a nut at each end. They are connected to the floor stringers as shown in Fig.

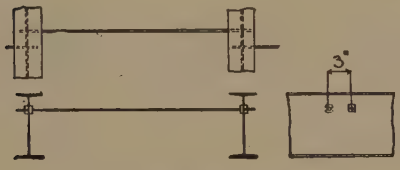


Fig. 137*a*.
Tie Rods and Beams.

Fig. 137*b*.
Proposed Arrangement of Rods.

137*a*. Their purpose is either to keep the top flange from buckling or to resist the thrust of the arch. For the former purpose, the spacing of the rods should not exceed twenty times the width of the flange. Holes for the rods should be placed as near the top as is practicable. This system, extending through a series of stringers, makes it necessary for them, or for part of them, to buckle the same way. They would seem much more effective to the writer if diagonal rods were introduced at intervals of not more than twenty beams, as shown in Fig. 137*b*. However, they might interfere seriously with some floor systems.

If the floor is an arch, the rods should be passed through the intersection of the web of the beam and the line of pressure for

* Values may be higher for Bethlehem beams.

the arch under full load. For a series of equal spans, the rods should have a net area sufficient to carry the maximum thrust of one span without exceeding the elastic limit. This may be obtained by the use of the formula,

$$H = wL^2/8R,$$

where H is the thrust of the arch in pounds per lineal foot;
w, the total load on the arch in pounds per square foot.
L, the span of the arch in feet;
R, the rise of the arch in feet.

The curve of the arch which is usually a circle, may without sensible error, be considered as a parabola. A property of the latter curve is that the tangent at the end, e, is equal to twice the ordinate, om, divided by the abscissa, em. The remainder will be clear from Fig. 137*c*.

In a series of varying spans, use the largest. The spacing of the rods should be such that the maximum stress in the interior beams due to the largest unbalanced thrust in connection with the vertical load does not exceed the elastic limit. The outside beams should keep within this same limit when carrying entire horizontal thrust and the weight. These high stresses are allowed on account of the fact that masonry of the arch undoubtedly helps to distribute the load. If the springing line of the arch is near either flange, it should be considered to carry everything; if in the middle, the whole beam is to be taken, that is, the moment of inertia about a vertical axis is to be employed. In practice, two widely different methods are in use. One is to analyze the floor as an arch, making no allowance for its distributing effect, a safe but rather expensive procedure. The other is to make the spacing of the $\frac{3}{4}''$ rods vary with the size of the beam disregarding entirely the horizontal reactions. Some floors, notably those of reinforced concrete or equivalent types, do not produce any thrust. This the author regards as a very substantial advantage.

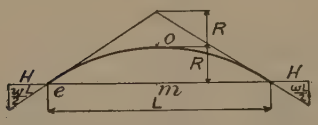


FIG. 137*c*.
Forces in a Parabolic Arch.

Connections

The floor stringers may frame into the floor girders,

(*1*) *Flush on top, Fig. 137d.*
(*2*) *Flush on bottom.*
(*3*) *An intermediate position.*

The latter is the cheapest in shop work as it avoids the coping necessary for the other two. However, (*1*) is the most economical as far as the floor is concerned and is the common type.

In any case, two angles make the connection as shown in Fig. 137*d*. For reasons given in Art. 58, the stringer is kept $\frac{1}{4}$ to $\frac{1}{2}''$ back from the girder. Use four-inch legs for one row of rivets and six-inch for two. $\frac{3}{8}''$ is the usual thickness, but $\frac{7}{16}''$ or $\frac{1}{2}''$ gives a better job. Standards, for example, the

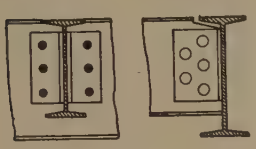


Fig. 137*d*.
Type (*1*).

Fig. 137*e*.
"Box Girder."

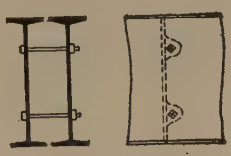


Fig. 137*f*.
Two Rolled Beams with Separators.

Carnegie, are used a great deal. They have some advantages, they economize templet work, allow of the use of the gang punch, and they may be prepared in advance. On the other hand, they frequently call for more rivets than are necessary and there is danger of using them for spans *less* than the minimum limit. Back to back of angles should be not over $\frac{1}{8}''$ less than the clear distance between beams.

For the connection of the stringers to the columns, either of the three types mentioned in Art. 126 may be used. It should be noted, however, that type (*2*), connection angles, cannot be employed with a closed section on account of the difficulty of driving the field rivets.

Sections for Short Spans, Floor Girders

For the usual 15 to 25 feet, we may employ,

(*a*) *One rolled beam.*
(*b*) *Two rolled beams.*
(*c*) *I-beams with plates riveted on.*
(*d*) *Two I-beams, riveted by plates top and bottom, a "box girder," Fig. 137e.*

(*a*) is best. For carrying walls or above the capacity of (*a*), (*b*) may be used. (*c*) is of somewhat doubtful economy. Even if the rivet holes are staggered at point of maximum moment, it would seem as though both should be deducted. (*d*) is a closed section and cannot be riveted to through the web, but must be bolted. The same objection usually applies to (*b*). Separators are used between the beams in the latter case to keep them a uniform distance apart, to equalize the loads, and to stiffen the flanges. They should be placed at reactions and every concentrated loading, but not more than 6 feet apart.

For walls, (*b*) is best on account of stability. In other locations, where stresses are too large for (*a*), use one of the forms mentioned under the next heading.

Sections for Long Spans, Floor Girders

Here we may employ,

(*e*) *Built I's, that is, plate girders.*
(*f*) *Built box girders.*
(*g*) *Trusses.*

Plate girders for buildings are much like those for railroad bridges, Chapter VI. Top flange should be about level with floor in order to be stiffened by it. An objection is the obstruction in the room below. For this reason, depth is often less than that determined by economy of material. This same reason may lead to the adoption of (*f*), although the same objection as for (*b*) and (*d*) still holds, and connections to the web must be bolted unless shelf angles are provided. Trusses, (*g*), are usually of the riveted Warren type, often with subverticals. In cases where the floor plan allows, trusses one or two stories deep save material. They are then concealed in

the partitions, their upper chord supports one floor; their lower, another; while the verticals may sustain an intermediate floor. Two angles, two angles and a plate, two channels rolled or built, with or without cover plate, are common for chords. One or two angles or two channels may be used for web members. Their construction varies little from that used in bridges, Chapter VIII. See Art. 147 for an example.

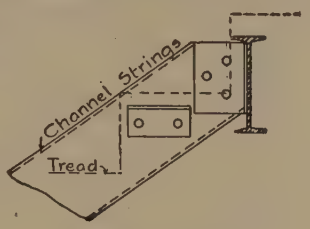


FIG. 137g.—Structural Steelwork for Stairway.

The most common case that calls for these long floor girders, is that where the owner wishes a large unobstructed room, often in one of the lower floors.

STAIRS

Stairways, Fig. 137g, consist of horizontal and vertical slabs (treads and risers), supported by two or three inclined beams (strings) attached to floor stringers or girders top and bottom. Three should be employed where the width exceeds four feet. The stringers may be of cast iron or structural steel. In the latter event, the treads rest upon horizontal angles fastened to channels, or occasionally, I beams. The treads may be of stone or cast iron; the former, while common, lacks fire-resisting qualities. Top of railing from top of tread in line with riser should be about three feet.

REFERENCES

Engineering Record, Vol. 58, p. 432. Use of Bethlehem Steel Sections.
Engineering News, Vol. 35, p. 251. Stairs.

EXAMPLES OF TRUSSES

Engineering News, Vol. 60, p. 609; Vol. 63, p. 294.
Engineering Record, Vol. 45, p. 470; Vol. 55, p. 495; Vol. 61, pp. 20, 186; Vol. 66, pp. 244, 430.

EXAMPLES OF PLATE GIRDERS

Engineering Record, Vol. 52, p. 71; Vol. 61, pp. 20, 186.
See also references of Arts. 133 and 147.

Prob. 137. Two adjacent segmental arches have spans of 8 and 12 feet and a common rise of 1 foot. Loads per sq.ft. including arches, are 100 lbs. dead and 70 live. The 24″ I @80# on the lower flange of which they rest has a span of 21 feet. Find maximum flexural stresses in same and in $\frac{3}{4}$″ tie rods, spaced 7 feet on centers.

Art. 138. Floors

We will use the term "floors" to cover the structure which carries the live load and its own dead weight to the stringer. Necessary qualities are,

(1) *Strength to bear these loads safely.*

(2) *Resistance to a temperature of 1700° F. for 4 hours while thus stressed.*

(3) *Ability to withstand fire streams after application of heat as above.*

Desirable qualities are,

(4) *Agreeable to stand upon.*

(5) *Acoustic top and bottom surfaces.*

(6) *Sound proof as far as possible.*

(7) *Light in weight.*

(8) *Durability.*

(9) *Economy.*

(5) and (6) Acoustics

A syllable spoken in a room radiates in all directions. On reaching the walls, it rebounds with a loss of energy varying with the nature of the surface. This loss is small with substances like steel, stone, or concrete; it is large with wood or plaster. Another factor to be considered is the effect of the various surfaces in causing the rebound to be concentrated in one or more spots or to be diffused. In general, the idea should be to have the sound pass directly from speaker to hearer; since the reflected waves of sound tend to confuse the direct waves, we reduce the former as much as possible. To do this,

(a) Use non-resonant material for external surfaces as far as practicable.

(b) Avoid vaulted roofs.

(*c*) Keep the height of the room proportionate to its size; a low ceiling gives less difference in time between direct and reflected air waves, hence reduced interference.

The best construction to prevent the passage of sound is the use of one or more enclosed air spaces with as little connection as possible between the walls. Next in value come those substances which contain a large number of air cells, such as mineral wool or plaster. The former is made of hot stone or slag, suddenly cooled by the application of water. It looks much like ordinary wool, but has the advantage of being fireproof.

(*2*) *and* (*3*). *Fireproof Construction*

The materials which satisfy these requirements are brick, terra cotta, tile, and concrete, or steel well protected by one of these. Terra cotta is baked clay. It may be either porous

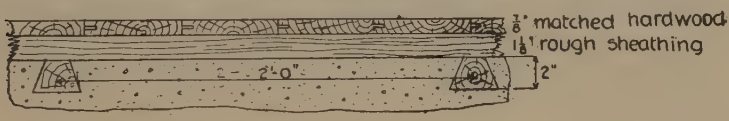


Fig. 138*a*.—Floor with a Wooden Top.

or hard burned, the former being made by mixing sawdust or straw with the clay before burning. It is lighter, more fire resisting, more expensive, but not as strong. Additional advantages for the porous are the facts that it is much more easily cut and that it may be nailed to.

The ordinary floor with the firetrap wooden joist is too dangerous to be seriously considered. However, the advantage of wood for reasons (*4*) and (*5*) are such that its use for the top of the floor is very common in spite of the fact that it will not survive a fire. Fig. 138*a* shows the usual construction. A concrete, granolithic, or tile surface may be used, but, except for its fireproof qualities, neither is as desirable as wood.

The common span is four to six feet.

Brick Arch Floors, Fig. 138*b*

Rise should never be less than one-eighth of the span and safe strength of brick masonry must not be exceeded. This value should be taken at about one-third the usual amount,

Art. 146, to provide for the variations in the line of pressure under live load. The common span is about 5 feet. There should be at least 2″ of concrete between the extrados of the arch and the bottom of the nailing pieces. This is a strong type but very heavy. It is highly fireproof itself but does not afford adequate protection to the beam. These two objections tend to prevent its use for high office work, but it is often specified for low buildings where the live load is heavy as in mills and warehouses.

Terra Cotta Arch Floors

Perhaps these are more typical of office work than any other floor. Here terra cotta in hollow blocks forms an arch set in cement. There are two common forms: the flat arch, Fig. 138*c*; and the segmental arch, Fig. 138*d*. Either one might be analyzed by conventional methods, but tests are generally

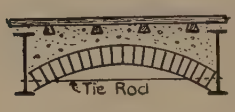


Fig. 138*b*.
Brick Arch Floor.

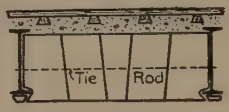


Fig. 138*c*.
Terra Cotta Flat Arch Floor.

Fig. 138*d*.
Terra Cotta Segmental Arch Floor.

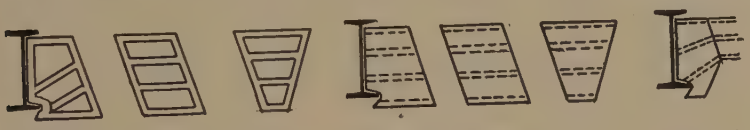
taken as a basis for the allowable loads. The flat arch would doubtless be more effective if joints were radial, but, on account of the additional expense, they are used only occasionally. The segmental arch is a much stronger type, allows a longer span, and is used in warehouses and mercantile buildings.

In the "side construction," the voids in the blocks run parallel to the stringers; in "end construction," they are perpendicular thereto. In the latter, all material is usefully employed, it has better fire-resisting properties, and the tie rods may pass through the voids. Fig. 138*e* shows typical members of each kind. However, there are many varieties. The side construction is sometimes used for the skewbacks, while the rest of the arch is end construction. Where the beam is deep, a raised skewback, Fig. 138*f*, may be used.

Generally a flat arch is made about the same depth as the beams but is placed 2″ lower. The space between top of arch

and bottom of flooring may be filled in by concrete or hollow terra cotta blocks, Fig. 138*g*.

The properties of terra cotta vary somewhat. For preliminary designs and approximate estimates, the following tables may be taken as guides. The effective depth is given in the first in order that thrust may be computed to determine size of tie rods. In the second, safe loads were obtained by the use of the arch formula and an allowable stress of 200 lbs. per sq.in. on net area of terra cotta. The rise varies between $\frac{1}{16}$ and $\frac{1}{6}$ of the span. For values not given in table, use the following rough rule. The safe load per sq.ft. for any given span will vary directly as the rise, that is, doubling the rise



Side Construction. End Construction.

FIG. 138*e*.

FIG. 138*f*. Raised Skewback.

doubles the load. The latter in both tables is safe amount of uniform live load from which the weight of the terra cotta itself has been already deducted. Cost will vary somewhat with job, time and location. Prices given are for large straight jobs in Pittsburgh, April, 1914.

TERRA COTTA FLAT ARCHES

Entire depth	6	7	8	9	10	12
Effective depth = rise inches	3.6	4.6	5.6	6.6	7.6	9.6
Weight in lbs. per sq.ft.	27	30	32	37	40	44
Cost in cents per sq.ft.	16	17	18	19	20	22
Safe loads in lbs. per sq.ft.						
Span, 4 feet	120	170	240	260	280	350
" 5 "	70	110	140	160	180	200
" 6 "	...	...	90	100	110	130
" 7 "	...	...	...	...	60	80
" 8 "	...	...	...	...	...	50

Segmental Terra Cotta Arches

Thickness arch, ins.	4		6		8		10	
Effective area, sq. ins.	28		36		43		47	
Weight in lbs. per sq.ft.	21		27		33		35	
Cost in cents per sq.ft.	18		20		22		24	
	Safe loads in pounds per square foot.							
Rise/Span	1/12	1/6	1/12	1/6	1/12	1/6	1/12	1/6
Span. 4 feet	920	1736	1184	2233	1414	2667	1545	2915
5 "	744	1379	957	1773	1143	2118	1249	2315
6 "	612	1148	788	1476	941	1763	1028	1927
7 "	520	983	669	1264	799	1510	873	1650
8 "	457	854	588	1099	703	1312	768	1434
9 "	403	759	518	977	619	1167	677	1275
10 "	359	683	462	879	552	1050	603	1147
11 "	327	617	421	794	503	948	550	1036
12 "	297	565	383	727	458	869	500	949
13 "	272	521	351	670	419	801	458	875
14 "	253	481	326	619	390	740	426	808
15 "	234	449	302	577	361	690	394	754
16 "	218	420	281	540	336	645	367	705
17 "	205	393	265	506	316	605	345	661
18 "	192	371	248	477	296	570	324	623
19 "	181	351	233	451	279	539	304	589
20 "	172	332	221	427	265	510	289	558
21 "	163	315	209	406	250	485	273	530
22 "	154	301	199	377	237	462	259	505
23 "	147	286	190	369	227	440	247	481
24 "	140	274	181	353	216	421	236	460

Combination Floors

These are really reinforced concrete floors with terra cotta fillers to lessen weight. As may be judged from the preceding and following tables, either of these types may be used for very long spans, thus eliminating floor stringers. There are a number of arrangements, one of which we give in Fig. 138*h*.

The floor as shown is ready for the screeds. Tables are based on construction shown in figure: concrete; one part Portland cement, two parts clean sand, and four parts stone; allowable fiber stresses in pounds per sq.in. for concrete in compression, 500; for steel in tension, 16,000. We give safe load in lbs. per sq.ft. on top of concrete layer.

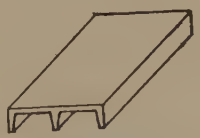


FIG. 138*g*.—Hollow Tile Filler.

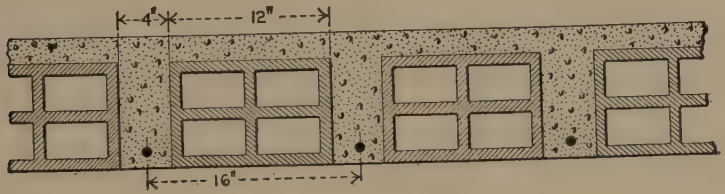


FIG. 138*h*.—Combination Floor.

Total depth, ins.....	6	7	8	9	10	11	12	14	17
Square steel rod, dia. ins.	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	$\frac{15}{16}$	$1\frac{1}{16}$	$1\frac{1}{8}$
Weight in lbs. per sq.ft..	50	55	60	65	70	75	80	90	105
Cost in cents per sq.ft...	21	23	26	28	31	34	36	40	50

Safe load in lbs. per sq.ft.

Span,										
5	feet	665								
6	"	446	660							
7	"	314	470	655						
8	"	229	347	487	650					
9	"	170	263	372	499	645				
10	"	128	202	290	392	509	640			
11	"	97	157	229	313	408	515	635		
12	"	74	123	183	252	332	421	521		
13	"	55	97	147	205	272	348	432	625	
14	"	41	76	118	168	225	289	361	526	
15	"	29	59	95	138	187	242	304	447	
16	"	...	45	77	113	156	204	258	381	610
17	"	...	34	60	93	130	172	220	328	527
18	"	...	...	48	76	108	145	187	283	459
19	"	...	...	37	61	90	123	159	245	402
20	"	...	...	...	49	74	103	136	212	352
21	"	...	...	...	38	61	86	116	184	310
22	"	...	...	...	...	49	72	98	159	272
23	"	...	...	...	...	39	60	83	138	240
24	"	...	...	...	...	30	49	70	119	212

Tile Floors

We will mention the Guastavino as an example. This is composed of three layers of tiling, care being taken to break joints. The tiles are about 6″ wide, 15″ long, and 1″ thick. This system is used largely in domes.

Most of the remaining floors are of steel in various forms, protected by concrete. The latter should be 1 part Portland cement, 2 parts sand, and 4 parts broken stone; or 1 part Portland cement, 1 sand, and 5 cinders. The former weighs 140 to 155 lbs. per cu.ft.; the latter, 80 to 110. Use about 1% of steel in the bottom of the slab to assist in carrying the moment, with shrinkage bars about $\frac{1}{2}$″ in diameter spaced alternately top and bottom every 18″ at right angles thereto.

An economical long span construction may be made of reinforced concrete joist about 5″×10″ and 2′ 0″ on centers. If a flat ceiling is desired the space may be filled with hollow tile or covered over with plaster on expanded metal or metal lath.

Fig. 138*i*.—Expanded Metal Floors.

Expanded Metal Floor

Here this steel is replaced by a metal sheet, slotted and spread out until it resembles poultry netting. Metal is placed near the bottom of the concrete; however, there must be not less than one inch of protection to prevent corrosion and damage by fire. There are three types, as shown in Fig. 138*i*. To determine approximately the strength of these systems, consider the steel as solid and of its original thickness. In the right-hand figure, the protection for the lower flange is made of plaster on expanded metal, supported by $\frac{3}{4}$″ channels or about $\frac{5}{8}$″ rods, spaced 12 to 16″ apart.

Roebling Floor

These are much like those for expanded metal and Fig. 138*i* might serve likewise for this type. It uses, instead of the expanded metal, a woven wire supported by rods. Although

this steel is designed as a form for the concrete, it undoubtedly assists in carrying the load, and it may be considered as reinforced concrete where its position and form justify it.

Columbian Floor

This is really a special type of reinforced concrete. The bars have different forms for different depths, see Fig. 138*j*. They are supported by clips punched to receive bars as shown in Fig. 138*k*. Figs. 138*l* and 138*m* show the finished floor, the latter having a suspended ceiling.

Fig. 138*j*.—Steel Reinforcing.

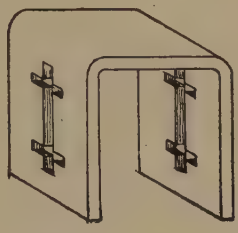


Fig. 138*k*.—Connection Clip.

Fig. 138*l*.

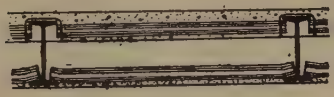


Fig. 138*m*.

Types of Finished Floor.

Metropolitan Floor

This differs from the preceding by utilizing a mixture of plaster of Paris and wood shaving instead of concrete. Fig. 138*n* shows the construction of this system ready for the nailing pieces. Exclusive of the latter and the beams, this type will weigh about 20 lbs. per sq.ft., to which 8 lbs. may be added for plaster and about 20 lbs. more for the wooden floor in case it is used. In addition to this 50 lbs., we find a capacity as follows:

Span in feet	5	6	7	8
Load in lbs. per sq.ft.	200	120	75	50

A large number of other systems are also used, but space will not permit a description. The one best fulfilling the requirements mentioned at the beginning of this article should be chosen. It might be well at this point to call attention to the advantage of using a type of floor which does not produce a thrust; also to the weakness of suspended ceilings in case of fire as considered in Art. 147.

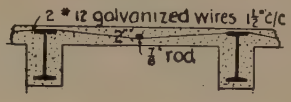


FIG. 138*n*.—Metropolitan Floor.

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Acoustics. Engineering Record, Vol. 41, pp. 349 et seq.
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Technograph No. 16, 1901–2, p. 5.
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Tests. Engineering News, Vol. 36, p. 298; Vol. 37, pp. 6, 215, 255, 269, 332; Vol. 38, p. 367; Vol. 46, p. 378; Vol. 47, p. 441; Vol. 70, p. 374.
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Engineering News, Vol. 34, p. 314; Vol. 40, p. 221.
Engineering Record, Vol. 58, p. 103. Terra Cotta.
American Society Civil Engineers, Vol. 34, p. 521. Terra Cotta.
Engineering News, Vol. 22, p. 434. Guastavino.
Engineering News, Vol. 41, p. 30. Expanded Metal.
Engineering News, Vol. 34, p. 45. Roebling.
Engineering News, Vol. 34, p. 333. Metropolitan.
See also references for Arts. 133 and 147.

Prob. 138. A 4″ brick arch has a span of 5 feet and a rise of 9″. Maximum allowable stress, 200 lbs. per sq.in. Find safe total load in lbs. per sq.ft.

Art. 139. Roofs

The requirements for roofing are much like those for flooring except as follows:

The live loads, save when open to the public, are much lighter, consisting of snow and wind only. The allowance should be same as for Mill Buildings, see Art. 131. The acoustic properties (*5*) of the top may be ignored, and (*4*) and (*6*) are replaced by the specification that it shall be water and weather proof. Protection against exposure fires is particularly important. Hence wood should not enter into the construction of roofs. Instead we find the following top surfaces:

(*A*) *Tile* roofs consist of several layers of waterproof roofing felt on which tile are embedded in cement. These

FIG. 139*a*.
A Book Tile.

FIG. 139*b*.
Book Tile Roofing.

may be of slate about 1″ thick or vitrified, $1\frac{1}{2}$″ in thickness.

(*B*) *Asphalt*, in two layers of $\frac{1}{2}$″ each.

(*C*) *Tar* and *Gravel*, much like Mill Buildings in its exterior surface. See Art. 124.

(*D*) *Brick*, set on edge in Portland cement on 5-ply paper.

(*E*) *Tin* or *Copper*. If used must not be placed in contact with concrete or terra cotta on account of corrosion.

These surfaces are supported by any of the systems of flooring mentioned in Art. 138, modified as necessary for the lighter loads. A common system involves the use of the *book tile*, Fig. 139*a*, which are supported on inverted T beams, 14 to 20″ center to center, Fig. 139*b*. A grave objection is the exposure of the steel. Even if covered by the plaster, there is insufficient protection for the T. The surface should have a slope of not less than $\frac{3}{4}$″ to the foot. There are four common methods of arranging the supports to the roof:

(*1*) *Pitch Roof*. Here the surface is supported by purlins,

usually of I-beams, borne in turn by trusses much like those for mill buildings, but varying in pitch with the architectural effect desired. The ceiling may be attached to I-beams running from truss to truss, or may be fastened to the bottom of the purlins. In the latter case, the trusses should be fireproofed, a rather troublesome job.

(2) *Beams Level.* The roof girders rest upon the columns, while the stringers run into columns or girders. Both are level. The ceiling is placed underneath exactly as for floors. The necessary slope in the roof is provided by building up with tile or concrete to the required level. The minimum distance between roof surface and steel is determined by the necessity for adequately protecting the steel against fire from above.

(3) *Beams Inclined, Ceiling Inclined.* Here the construction is similar to that for the floor except for its inclination and the

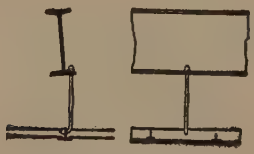

FIG. 139*c*.
Suspended Ceiling.

FIG. 139*d*.
Attachment to I-beam.

substitution of roof surface. This inclination is accomplished by using varying lengths of top story columns, still keeping stringers at right angles to girders. For warehouses and factories, the resulting inclined ceiling is not objectionable, but for mercantile or office buildings must be replaced by one of the other types.

(4) *Beams Inclined, Ceiling Level.* The roof surface and its support remain like (3). The ceiling, where closest to the roof, must allow of sufficient protection to the steel stringers and girders. Fig. 139*c* shows a typical arrangement. Bent clips, Fig. 139*d*, may be used in place of the rods of Fig. 139*c*, and bars instead of T beams. To these are wired the expanded metal, wire lath, or metal lath, on which is placed the plaster. It might be well at this point to call attention to the defects of a suspended ceiling as discussed in Art. 144.

REFERENCES

Example of Roofs, Engineering Record, Vol. 64, p. 164.
Engineering and Contracting, Vol. 40, p. 220.
See also references for Arts. 133 and 147.

Prob. 139. Assuming purlins spaced 6 feet or less apart on centers (measured horizontally), design roof of building of Prob. 136 for a total load of 100 lbs. per hor. sq.ft., and compare weights for the different methods. Material medium steel. Building 60 feet square. Use allowable stresses of Art. 146.

Art. 140. Cornices

The projection which usually occurs at the intersection of the roof and wall is called the cornice. It is occasionally

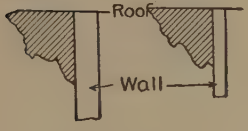

Fig. 140*a*.
Typical Cornices.

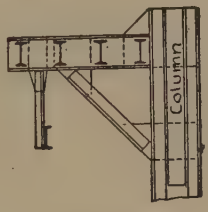

Fig. 140*b*.

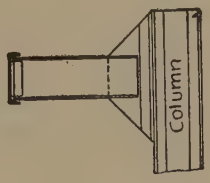

Fig. 140*c*.
Steel Supports for Cornice.

Fig. 140*d*.
Steel Support.

Fig. 140*e*.

Fig. 140*f*.
Clamps.

employed as in mill buildings to conduct away the water However, its main purpose is architectural, that is, to add to the appearance of the building. We shall not take up this phase of the subject, but will give typical examples in Fig. 140*a*.

Structurally, it is quite disadvantageous. It is much exposed to attack by the flames, hence should be carefully fireproofed; it is an eccentric load on the columns; the arrangement of the

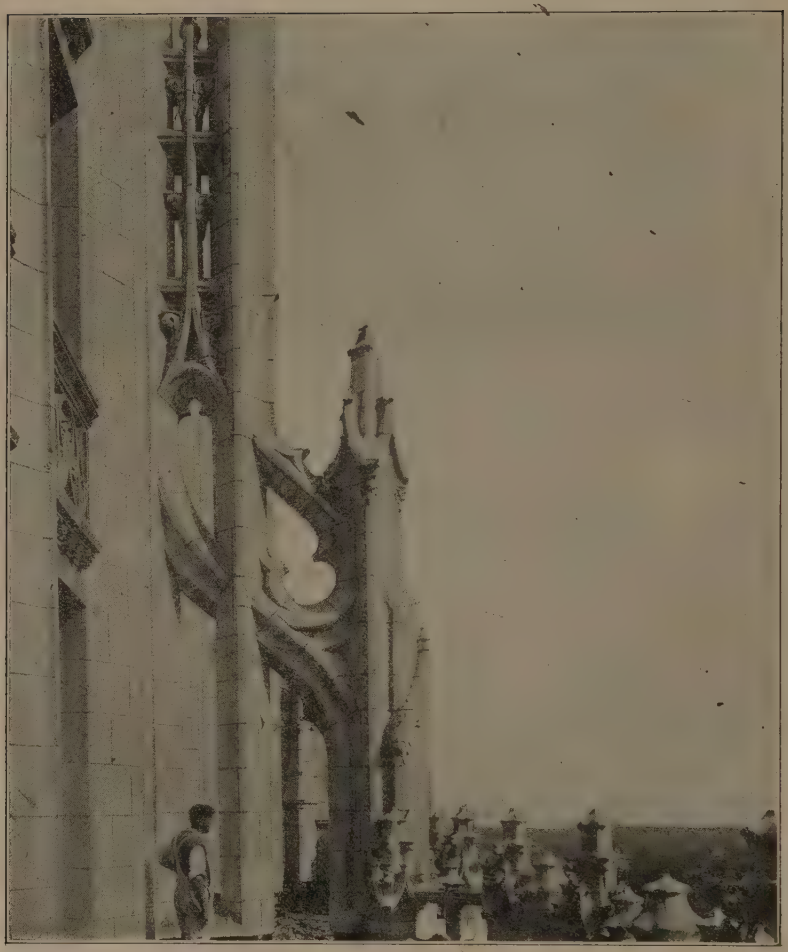

FIG. 140g.
Examples of Ornamental Terra Cotta. Atlantic Terra Cotta Co., New York.

steel to transfer weight to the latter is awkward; and great care must be taken to secure it to the frame work. Notwithstanding these objections, one should be used on all exposed surfaces of the building, as it adds much to its appearance.

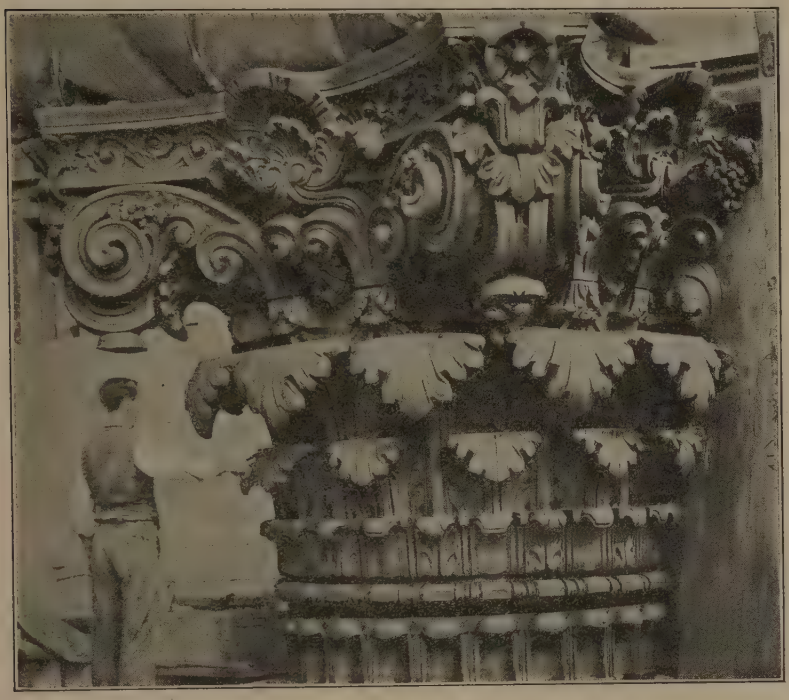

FIG. 140*h*.

Examples of Ornamental Terra Cotta. Atlantic Terra Cotta Co., New York.

The supports for the cornices are usually beams carried by brackets at each column or on cantilevers. Figs. 140*b*, 140*c*, and 140*d* show typical details. This steel should be punched at intervals to provide fastening for the clamps as explained below.

Cornices are usually made of blocks of terra cotta or stone. The latter often presents the better appearance, but it is heavy and lacks fire-resisting qualities. These blocks should be

laid in cement and enough of them firmly fastened to the steel to make the whole secure. Clamps, bolts, hook bolts, or dowels, Fig. 140*e* and 140*f*, may be used, $\frac{1}{4}''$ being a common

Fig. 140*i*.
Examples of Ornamental Terra Cotta. Atlantic Terra Cotta Co., New York.

size. It takes three fastenings to hold a block rigidly in position, but less will generally do, since adjoining blocks and the bond of the cement help a good deal. Something depends on the amount and boldness of the projection. Particularly

efficient connection to the steel is advisable in localities subject to earthquakes.

Prob. 140. A roof column 15 feet long between supports has a section of one Pl. $11'' \times \frac{3}{8}''$ and four L's $6'' \times 4'' \times \frac{3}{8}''$, $11\frac{1}{4}''$ back to back of the 6″ legs. Beams framing into these legs on either side have loads of 5000 lbs. apiece. On one other side against the 4″ legs is a load of 15,000 lbs. On the fourth side is the cornice weighing 5000 lbs. with its center of gravity 2 feet from column. Find direct and flexural stresses,

(*a*) when loads are all dead.
(*b*) when loads are half dead and half live.

NOTE. It would be very desirable to prevent these large flexural stresses by designing joints to transmit the bending to opposite beams. Assume, however, that this has not been done.

Art. 141. Wind Bracing

As its name implies, the principal function of the wind bracing is to resist the thrust of the wind. In addition, it gives stiffness to the structure, and, if properly designed, will resist earthquake shocks.

It is not necessary that this bracing fill every panel in the building. For example, in the floor plan of Fig. 141*a*, complete systems of bracing at points shown is ample, the usual types of floors giving stiffness to those columns not directly braced. A well-built solid masonry wall will help very much, and for heights less than twice the least base, they may be considered equivalent to wind bracing in their own plane. Tile partitions also assist, but should not be considered in computations. Another aid is the stiffness of connections between stringers, girders, and columns. There is some divergence in practice on this subject. Some engineers omit all provision for wind except for tall narrow buildings. The author believes that except as indicated above for walls, all high office buildings should be so designed that the addition of wind to other loads would not cause stresses more than 25% in excess of ordinary allowable values.

There are two general types, *X bracing* and *portal bracing*. The former is better and more economical and should be used unless it interferes too much with the construction of the building.

X Wind Bracing

This may be designed with *adjustable members*, with *stiff tension members*, or with *compression members*, Art. 96. The first may be of eyebars, clevis rods, or loop rods. Each of these calls for pins at each end. If two channels are employed for the strut, which also often acts as a beam to carry the vertical loads, a pin may be passed through them to carry the bars, Fig. 141*b*. For a single beam we may use the detail seen in Fig. 141*c*. This will require one rod with clevis nut or with

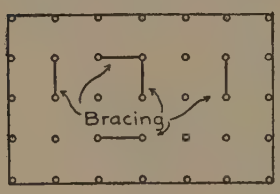

Fig. 141*a*.—Location of Wind Bracing.

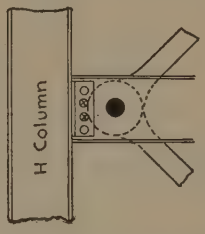

Fig. 141*b*.—Eyebar Bracing. End Details.

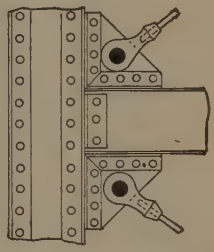

Fig. 141*c*.—Bracing of Rods with Clevis Nuts.

double loop, or two ordinary loop rods or eyebars. It will be noted that a much better intersection of the axes is secured in the second case, but an objectionable tension on the rivets is present. The angle or channel bracing is more modern and is made much the same as for viaducts, Art. 96, being attached to plates like those shown in Fig. 141*c*, except that they are likely to be larger and rivets must be substituted for the pin. As in other places, channels are best but more expensive; rods are poorest and cheapest.

Portal or Knee Bracing

This carries the shear by the stiffness of the beams and columns. Sometimes the beams between the braced columns are simply made deeper, a plate or lattice girder being common. Additional stiffness is secured by the insertion of a knee brace,

Figs. 141*d*, 141*e*, and 141*f*, arranged as in Fig. 141*g*. The upper braces may be used only in walls or partitions. The farther out from the column, the greater the efficiency of the bracing.

In a high narrow building where comparatively few columns are assumed to carry the load, there is often a possible tension on the windward side. Columns can be anchored by means of bolts extending into the masonry, or trusses may be inserted in the basement, anchoring to adjoining columns.

Computations

To obtain wind stresses in the typical cage construction:

(*1*) Wind must be considered as acting on either face of the building; for the usual rectangular structure, there are only two cases.

Fig. 141*d*.

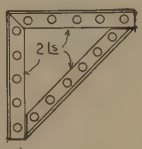

Fig. 141*e*.

Details of Knee Braces.

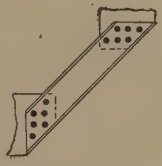

Fig. 141*f*.

Fig. 141*g*. Arrangement of Knee Braces.

(*2*) Compute total wind load at each floor.

(*3*) Decide on columns between which there will be bracing. This is determined partly by convenience to tenants, and partly by economy. If there is portal bracing, it is best to utilize all columns in both directions, if practicable; for X-bracing, to use enough so that section of columns will not have to be increased.

(*4*) The centers of gravity of loads and resistances should coincide, and forces assigned to various systems of bracing ought to vary with their stiffness. The exact distribution of these wind stresses among the braced columns may be obtained by equating deflections or by the method of least work. As these are quite tedious and complicated, it is customary among engineers to assign certain portions of the wind to each set of bracing, making it agree, as closely as possible

by eye, with above named conditions. In competent hands, this should secure results sufficiently accurate. Notice that if systems of bracing are located far apart, floor must be relied upon to carry the load. Sometimes horizontal bracing is employed for this purpose, or to transfer wind shear from one system to others not directly below, or to stiffen heavy trusses or girders. Plates riveted to beams or clevis rods may be employed for the diagonal members.

If the braced systems are of the X type, their treatment as simple cantilever trusses involves no difficulty and may be readily followed in problem of Art. 147. But for knee bracing we must proceed further. See Continuous Portals, Art. 99.

(5) We assume that the total shear on any story of any system is equal to that system's share of the loads above the

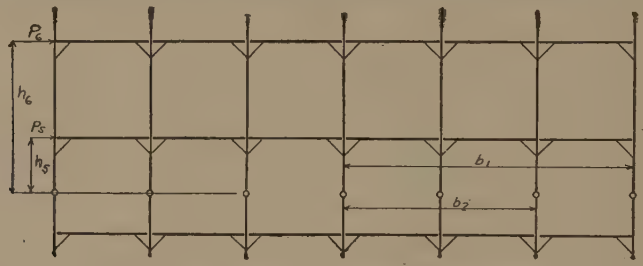

FIG. 141*h*.—Forces in a Knee-braced Building.

story and that this shear is divided among the columns in proportion to their moments of inertia.

(6) We compute center of gravity of column areas in each section. If then we pass a section through the point of contraflexure in a story, half way between the supports of the column, Fig. 141*h*, the shear is divided among the various columns in proportion to their moments of inertia. Also the moment above said section is carried by fiber stresses which are proportional to their distances, b_1, b_2, and so forth, from the neutral axis, that is,

Area of Col. (1) × unit stress in Col. (1) × b_1
plus
Area of Col. (2) × unit stress in Col. (2) × b_2
plus
and so forth.

$= P_5h_5 + P_6h_6$ + and so forth

An equation which may be readily solved when we remember that the unit stresses vary as their distances from the neutral axis.

Example

As a simple example, let us find wind stresses on eighth floor of twelve story system with loads and dimensions as shown in Fig. 141*i*. Let the columns at this point be Bethlehem H sections, 12″ a 78#, placed with webs perpendicular to plane of paper for outside columns, and parallel thereto for the remainder.

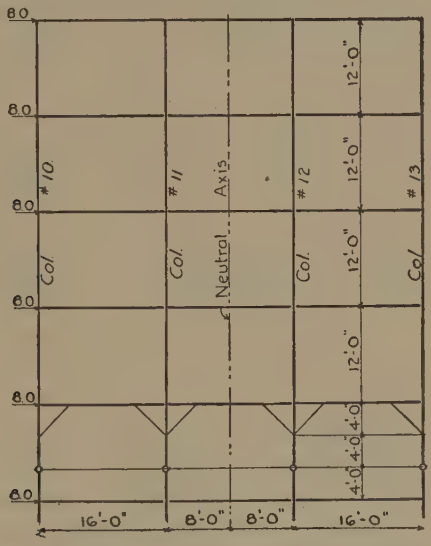

Fig. 141*i*. Illustrative Knee-braced Building.

Shear in the eighth story is 40.0 kips.

Total moment of inertia of columns is $208+616+616+208=1648$ in^4.

Shear on each exterior column is $\frac{208}{1648}\times 40.0 = 5.05$ kips.

Bending moment for same, $5.05\times 48 = 242.4$ kip in.

Fiber stress due to flexure $= \frac{M}{S} = \frac{242.4}{34.7} = 7.0$ kips per sq.in.

Shear on each interior column is $\frac{616}{1648} \times 40.0 = 14.95$ kips.

Bending moment for same, $14.95 \times 48 = 718$ kip in.

Fiber stress due to flexure $= \frac{718}{102.6} = 7.0$ kips per sq.in.

Moment of loads above the eighth story is,

$$8(8+20+32+44+56) = 1280 \text{ kip ft.} = 15{,}360 \text{ kip in.}$$

Center of gravity is half way between columns (11) and (12).

Let C be the stress in outside column in kips per sq.in., and A the common area of each column $= 22.94$ sq.in.

$$288CA + 96CA/3 + 96CA/3 + 288CA = 15{,}360 \text{ kip in.}$$

whence $C = 1.05$ kips per sq.in., direct wind with stress in outside columns.

$$\frac{C}{3} = 0.35 \text{ kips per sq.in. in inside columns.}$$

Total wind stress in outside columns, $7.0 + 1.1 = 8.1$ kips per sq.in.

Total wind stress in inside columns, $7.0 + 0.4 = 7.4$ kips per sq.in.

Shearing unit stresses are usually unimportant and consequently ignored.

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Ohio State University Bulletin for June, 1912, No. 40. Vol. XVI. "Stresses in Tall Buildings," by Cyrus A. Melick.

American Society Civil Engineers, Vol. 33, p. 190; Vol. 37, p. 221.

Engineering News, Vol. 55, p. 420; Vol. 57, p. 138; Vol. 63, p. 294; Vol. 69, p. 492.

Engineering Record, Vol. 58, p. 272; Vol. 66, p. 165.

See also general references, Arts. 133 and 147.

Prob. 141. Let Fig. 141*a* represent a section of a building of 10 stories of 12 feet each. Spacing of columns, 16 feet in either direction. Loads: 100 lbs. per sq.ft., live and an equal amount dead for both roof and floors; wall, 1500 lbs. per lin.ft. Wind, 30 lbs. per sq.ft. with a full panel load on top. Use knee bracing of 3 feet effective depth. Design columns in first story, using H sections. State both wind and direct stresses. Use Specifications of Art. 146.

Art. 142. Exterior Walls

The purpose of these walls is to exclude rain and weather, to retain the heat, and to protect against fire. At the same time openings must be provided for the admission of light and air (windows) and for entrance (doors). These conditions are satisfied by four materials:

(*A*) *Terra Cotta Masonry.*
(*B*) *Brick Masonry.*
(*C*) *Stone Masonry.*
(*D*) *Concrete Masonry.*

The last is quite common in buildings entirely of concrete and is usually reinforced; however, it will not be considered here. Both (*A*) and (*B*) are good and are extensively used. (*C*) is expensive and lacks fire-resisting qualities.

Type (*3*) as mentioned in Art. 133 is usual in high office buildings. The thickness of the walls of any type at any point should not be less than

$$\frac{\text{Height of that point above its support}}{50} \text{ plus } 6''.$$

Supports for Walls

The outside columns are commonly set in the wall a little inside of the center. The protection for the outermost steel should be not less than $4''$ and more is desirable. An ideal theoretic position is such that the center of gravity of the columns coincides with applied loads.

The beams which carry the wall should not only be amply strong, but also stiff to prevent the cracking of the masonry. While the latter possesses considerable strength of its own, it is best to neglect this. With the masonry load so considered the deflection should, under no consideration exceed one 360th of the span and it ought to be less. Violation of these rules is likely to result in cracked walls. See Art 148.

I-beams, rolled or built up, and channels act as supports. The use of two instead of one is common since it tends to increase stability. For the same reason, a plate riveted to the bottom of one I-beam or two channels may be employed.

When wall beams are parallel to the stringers, they must be so placed that they support the flooring. When parallel to the girders, their position is determined by the necessity for supporting the stringers. In the former case, an angle may

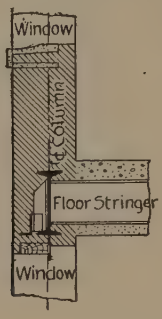

FIG. 142*a*.

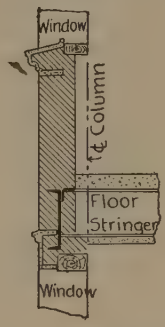

FIG. 142*b*.

FIG. 142*c*.

Sections of Typical Walls.

be riveted on if it is desired to keep the bottom of the beam below the level of the stringers, Fig. 142*d*. In the latter event, it is desirable, especially for double beams, to place the stringers on top of the wall beams; as the former are stiffened by the masonry the usual objections to this connection are not valid. If two beams are employed, they should be united by separators or by plates to form a box girder. They must also be planned to give adequate support for any anchors which may be necessary. Often, small beams may be used for this purpose as seen in Fig. 142*e*.

FIG. 142*d*.
Intermediate Support for Floor.

FIG. 142*e*.
Anchor Beam.

Windows are much the same as for mill buildings. The trim, however, is usually terra cotta, and metal often takes the place of wooden parts on account of its greater security against fire.

Ordinary doors will be described in the next article. The main entrance should be treated architecturally, and its design properly forms no part of this work.

Bay Windows

(*a*) T beams for floor above and ceiling below.
(*b*) Brackets.
(*c*) Stringers.
(*d*) Wall Beams.

The cage construction readily allows of bay windows. These are supported by cantilever brackets or beams, resting on

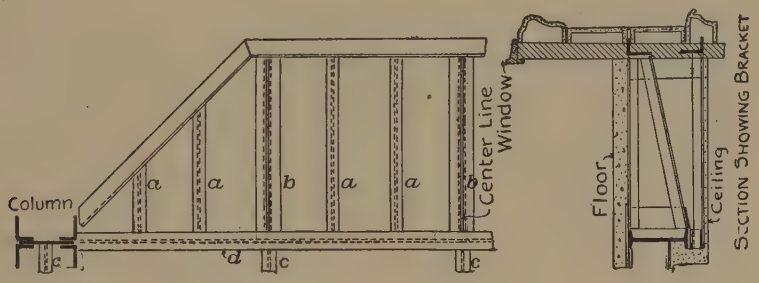

Fig. 142*f*.

columns or girders, preferably at points where the stringer connects on the other side. Great care must be taken to carry the large twisting moment of these loads. If the cantilever is not opposite another stringer, the beam supporting them must be designed for the torsion. Also the projection of the cantilever must not be enough so that under any circumstances, live plus the dead on the bay window can overturn the dead alone on the other span.

For general references, see Arts. 133 and 147.

Prob. 142. Using above rule for the thickness of wall, derive an expression for the minimum total volume for any given height h.

Art. 143. Partitions

The objects of partitions are:

(1) To ensure privacy.
(2) To retard fire.

In addition it is sometimes required:

(3) To allow passage.
(4) To admit light.
(5) To carry a load.

The last object is best fulfilled by the brick partition, usually 8 to 12″ in thickness. However, on account of its large weight, it is cheaper and much more common to use steel columns for this purpose. (3) calls for doors and (4) for windows, glazed to obstruct vision. Both interfere with the fire resisting qualities, the latter so seriously that they should be avoided except where absolutely necessary. Doors are made by nailing to the rough lumber, a frame, *ab*, as shown in Fig. 143*a*.

There are four common types of partitions in high office buildings:

(1) *Solid Plaster.*
(2) *Hollow Plaster.*
(3) *Tile.*
(4) *Gypsum Block.*

For (1) and (2), vertical supports known as studs must be provided. These may be bars, angles, or channels, the latter being the most common. The spacing is 12 to 18″ on centers, decreasing with the height. The largest dimension of the steel shapes mentioned above lies between $\frac{3}{4}$″ and 4″, the smaller sizes being more common. They are often attached to a nailing strip, which in turn is fastened to the ceiling and floor. This is objectionable, however, from a fireproofing point of view, and it is better to nail or screw directly. For the larger shapes, an angle is provided for this purpose; others may be bent and the holes for nails or screws punched in the horizontal portion.

The hollow partition is made by placing wire lath or expanded metal on both sides of the studs. The thickness of the plaster

must not be less than $\frac{1}{2}''$. By using $\frac{3}{4}''$ channels, the total thickness may be reduced to $2''$. In the solid partition, the interior space is filled with plaster, and for a thin partition one sheet of wire lath may be omitted. The weight of these partitions varies with the details. Roughly, letting t represent the thickness in inches and w, the weight in lbs. per sq.ft.,

$$w = 8t \quad \text{for solid partitions.}$$
$$= 15 + 2t \quad \text{for hollow partitions.}$$

Tile Partitions

Tile partitions are made of terra cotta blocks. They are laid like bricks to form a partition, except that they are usually placed on edge and that a single row makes a wall. For such a partition, the maximum distance between supports either vertically or horizontally should not exceed forty times the thickness. 60 lbs. per cu.ft. may be allowed for the weight.

Fig. 143*a*.
Frame for Door.

Fig. 143*b*.
Solid Plaster Partition.

Tile should be placed directly upon the floor. Blocks which will be nailed to in forming the finish should be porous and contain large air spaces. Or wooden blocks may be employed. The partitions may be set beginning at the top story and each one tightly wedged by slate, or we may commence at the bottom floor and work up, the added deflection tightening the partition. An objection to the latter is that too much pressure at the bottom may cause cracks, while at the top there is not enough to hold it in place.

Gypsum Block Partitions

These are laid somewhat like the preceding. The blocks are about $15'' \times 27'' \times 2'$ to $6''$ in thickness. They are made of sawdust and gypsum, and may be cut with a wood saw. They are cheaper and lighter than tile, but not as strong.

REFERENCES

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Tests.—Engineering News, Vol. 46, p. 482, 489; Vol. 68, p. 403.
Engineering Record, Vol. 66, p. 487.
Engineering Magazine, Vol. 45, p. 853; Vol. 46, p. 53.
See also general references, Arts. 133 and 147.

Prob. 143. What is the stress due to its own weight in a solid concrete partition 12 feet high and 20 feet between supports? Weight per cubic foot is 80 pounds.

Art. 144. Steel Protection

To ensure permanency of the building and the safety of its occupants, it is imperative that adequate provision be made against the destruction of the building either by slow corrosion or by fire.

Rust Prevention

As explained in Art. 13, rusting is the union of iron and oxygen in the presence of water and carbonic acid. The absence of either one of the three foreign elements will protect the steel. Cement mortar, properly laid, allows the water to penetrate but slowly and its alkaline nature neutralizes the acid, hence renders rusting impossible. This conclusion has been verified by many observations which tend to show that metal protected by $\frac{3}{4}''$ of cement or cement concrete is well preserved. Stone is a better aggregate than cinder, as the latter often contains substances like sulphur and oxide of iron, which corrode the steel. Lime mortar probably acts the same way; but there is considerable doubt if it is as efficient as cement.

If the steel is to act structurally with the cement, it should not be painted; if the cement is simply a protection, it may be painted or not; if cement is not in contact, the steel must be painted. Under the latter condition, rust is to be expected, although experience indicates that it will take place much more slowly than in the open air.

Fireproofing

(*1*) A fireproof building is one where the members on which the safety of the structure depends would retain their strength during a severe fire. Every building exceeding five stories in height should comply with this requirement. In addition, it is desirable:

(*2*) That the damage to the other parts of the building, either by fire or water, be reduced to a minimum.

(*3*) That the building be so constructed as to resist the spread of fire and, if possible, to confine it to the room where it originated.

(*4*) That the exterior be such as to resist fires in other buildings, the so-called exposure fires.

The contents of most buildings include sufficient combustible material, which, if once kindled, will severely try its fireproofing qualities. This temperature may rise as high as 2500 degrees F. Now the strength of steel decreases nearly uniformly from 600 degrees F., at which it retains its full strength, to 1600 degrees, at which it is nearly zero. Brick, terra cotta, and concrete, the former two laid in cement, have proven superior for the protection of steel. Stone is weak in fire-resisting qualities and cinder concrete is much the better type in fireproofing work. Protection from the effects of a fire hose must also be considered. The impact of the stream or the sudden change of temperature may cause damage.

Protection for Beams

There are three methods in common use:

(*1*) *Suspended Ceiling.*
(*2*) *Terra Cotta.*
(*3*) *Concrete.*

Type (*1*) is shown in Figs. 138*m* and 139*c* and explained by adjoining text. As ordinarily built, this has been found to be entirely insufficient for fire protection; it should therefore be used only when one of the other systems is applied to the beams above. (*2*) is shown in Figs. 138*e* and 138*f*;* the beam ought

*A little trapezoidal piece not shown in the cut protects the bottom of the beam.

not to be nearer the edge than 2 to 3″, varying with the importance and exposure of the beam. The terra cotta protection should be of the porous variety; the hard burned is likely to crack under successive applications of heat and water. (3) is shown in Figs. 138*i* and 138*l*; here the protection should be at least 3″ to 4″, varying as for (2).

Protection for Columns

Here three similar systems are used. However, more care must be taken than in beams on account of the importance of these members and their exposure on all sides. System (1), which consists of a single or double layer of metal lath and plaster, cannot be depended upon to withstand a severe fire and the application of water. Under these circumstances, it is likely to break off and expose the columns, thereby increasing the cost of the repairs and endangering the building.

Fig. 144*a*.

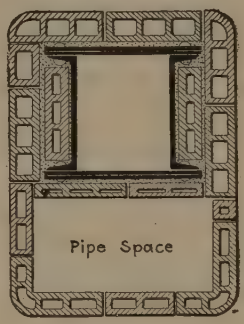

Fig. 144*b*.

Terra Cotta Column Fireproofing.

Terra cotta protection for columns should be of the porous variety. The common construction consists of a shell 2 to 4″ thick, containing an air space independent of the main column, Figs. 144*a* and 144*b*.

There are two ways in which concrete may be used for protection. In the first method, a hollow casing is formed around the column; in the second, the latter is embedded in the concrete. The writer prefers the latter and would also fill all the interior spaces, thus protecting the steel. In general there should be a minimum protection of about four inches, one of which may be an air space. Single projecting

parts of the steel may be allowed to approach an inch nearer the edge, as for example, in a square column with a circular casing. See Figs. 144*c* and 144*d*.

If brickwork be employed, there should be at least one course for the protection of steel in a wall; and two if in a room. Both the brick and the terra cotta are set in mortar.

As already noted in Art. 137, stairways are often unprotected and made of material which lacks fire-resisting properties. In view of their function as given in Art. 134, this seems a mistake. Other points to be particularly borne in mind are:

(*a*) Avoid nailing strips in partitions and fireproofing; they burn and allow the whole to fall.

FIG. 144*c*.

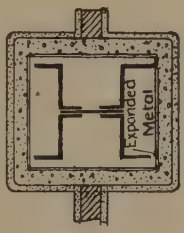

FIG. 144*d*.

Concrete Column Fireproofing.

(*b*) Keep pipes outside of the column casing.

(*c*) Arrange to confine fire as far as possible to room and floor where it originates; particularly, avoid vertical openings communicating with each floor.

(*d*) Give exposure fires much attention. Unless specially designed to resist these, a fireproof building may suffer severely from fire in an adjoining combustible structure. Windows are the weak points in attacks of this sort. Wire glass and fireproof shutters are the best protective measures. The trouble with the latter is that they are likely to be left open.

(*e*) Design with extreme care the fireproofing for the supports for heavy loads such as safes, water tanks, and so on. Arrange for an abundant supply of water for fire-fighting purposes as considered in the following article.

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Engineering News, Vol. 35, p. 234 et seq.; Vol. 40, p. 278; Vol. 50, p. 343; Vol. 52, p. 265; Vol. 53, p. 574; Vol. 59, p. 627.
Engineering Record, Vol. 39, p. 261; Vol. 55, p. 698.
Engineers Society Western Pennsylvania, Vol. 18, p. 226; Vol. 27, p. 55.
See also references for Arts. 138 and 143.

Corrosion. Engineering News, Vol. 57, p. 124.
Engineering Record, Vol. 47, p. 394.

Tests. Engineering News, Vol. 41, pp. 360, 364; Vol. 42, p. 308; Vol. 55, pp. 115, 116, 128, 603.
U. S. Geological Bulletin No. 370.

Fires. Engineering News, Vol. 28, p. 511; Vol. 37, pp. 313, 316; Vol. 40, p. 366; Vol. 43, p. 266; Vol. 51, p. 220; Vol. 53, p. 136 et seq.; Vol. 55, p. 471 et seq.; Vol. 56, p. 136 et seq.; Vol. 59, p. 73 et seq.; Vol. 63, p. 153.
Engineering Record, Vol. 39, p. 24; Vol. 49, p. 177 et seq.; Vol. 50, p. 134 et seq.

Art. 145. Requirements for Plumbing and Mechanical Plant

The design of the structural part of the building is closely interwoven with the mechanical, electrical, and sanitary portion. It must not be forgotten that financial success is largely dependent upon the latter three. The tenants of a modern office building demand the following conveniences:

(1) *Ordinary Water.*
(2) *Hot Water.*
(3) *Drinking Water.*
(4) *Fire Hose.*
(5) *Toilet Room.*
(6) *Waste Disposal.*
(7) *Elevators.*
(8) *Ventilation.*
(9) *Heat.*
(10) *Light.*
(11) *Telephone and Telegraph.*

The field is entirely too broad for us to attempt to do more than give a few fundamental facts concerning each.

(1) *Ordinary Water*

This usually comes from the city mains. In some cases an artesian well supply or the drainage water, filtered if necessary, may be cheaper. The amount for all purposes can be estimated at $\frac{1}{4}$ to $\frac{1}{2}$ gallon per sq.ft. per day. It is used in the boilers, condensers, air washers and elevators; for closets, urinals, and slop-sinks; for hot water, drinking water, and water for fighting fire. Unchanged it appears at one or two faucets at each wash basin, one of which should be located in each suite. In the toilet rooms, there should be about one-third as many basins as the combined number of urinals and closets. In a suite there should not be less than one-fifteenth the number of the expected occupants. The size of these basins is usually 18″ to 2 feet square.

There are two ways of maintaining a pressure greater than that in the mains, which is usually insufficient—by continuous pumping or by the use of tanks located on the roof, with an air space to allow of a higher head than that furnished by gravity. The latter, while more expensive at first, lessens cost of operation. For very high buildings, a double system may be used, one for the upper floors, and one of lesser head for the lower floors. The laws of Hydraulic Design suffice for the determination of pressure head and sizes of pipes.

(2) *Hot Water*

The preceding may be heated by live or exhaust steam as discussed under (9). Its use is confined to washstands.

(3) *Drinking Water*

This is sometimes omitted, particularly if the city supply be of good quality. If not, it may be filtered, distilled, or obtained from artesian wells. It is often cooled by means of an ammonia condenser in the basement with an expansion coil around the supply pipe near the outlet. These outlets are faucets, usually one to each floor.

(4) *Fire Hose*

The number of fire streams of 150 gallons per minute capacity advisable for any floor may be obtained by dividing the floor area in sq.ft. by 5000. The number for any entire building should be equal to the entire floor area divided by 40,000 plus 3 additional. The hose should be centrally placed in a conspicuous location and be in such lengths as to cover all parts of the building. The pump which furnishes the pressure should be chosen for its reliability and simplicity rather than economy of operation. Connection for city fire engines should also be provided. These requirements will determine necessary sizes when applied with the aid of the principles of Hydraulic Design.

It is often assumed that no fire can take place in a fireproof building, and provision for such a contingency consequently omitted. However, as shown in the preceding article, this reasoning is fallacious.

(5) *Toilet Rooms*

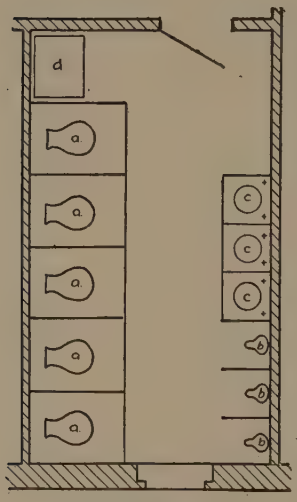

FIG. 145*a*.
Typical Toilet Room.

These were discussed in general in Art. 134. Fig. 145*a* shows a typical arrangement which will care for about 20,000 sq.ft. of office space.

Plumbing practice dictates the following sizes:

	Waste	Vent
a, Closets	5″	4″
b, Urinals	2	2
c, Washstands,	1½	1½
d, Slop Sink	3	2

(6) *Waste Disposal*

These pipes, both vent and waste, run to the nearest main. The latter are arranged together with the mains of other systems in groups, which are often placed either inside or around a column. They are commonly of cast iron and an average diameter is six inches. In a similar manner, the rain from

the roof is conducted down inside the building. The location of these groups of pipes and of the toilet rooms should be such as to promote economy. This is usually obtained by placing the toilet rooms in vertical tiers.

(7) *Elevators*

Confining ourselves at the start to the very important feature of passenger elevator service, we find two essential characteristics:

(*a*) Time interval. (*b*) Capacity.

(*a*) is the longest time which a man may have to wait for a car. Taking the case where they stop at all floors, the time interval in seconds is equal to:

$$\frac{\text{Height of travel in feet}}{\text{Rated speed in ft. per min.}} \times \frac{C}{\text{number of cars}}.$$

If it were possible for a car to stop and start instantaneously running at full speed between floors, the value of C would be 120. This theoretic limit is evidently impossible, a mean value in practice being 330. A higher value may be obtained in rush hours, or be caused by a lack of signal bells or inefficient management. A low time interval of say 20 seconds is desirable, but it will be found expensive in a high building of small area. Alternative schemes, such as elevators express from the first to the tenth floors, or elevators going part way up, are sometimes used.

The number of elevators necessary for an ordinary office building may be obtained by multiplying entire area of all floors above first in square feet by height of elevator travel in feet and dividing by 200 times rated speed in feet per minute times area of elevator in square feet.

A typical diagrammatic view of the hydraulic elevator is given in Fig. 145*b*. The weight of the car may be estimated at 100 lbs. per sq.ft. of floor space; the counter weight is made about three-fourths of this. The elevator falls freely under this unbalanced weight; it goes up on the admission of water from the pressure tanks. Speed should be about 400 to 500 feet per minute in either direction. The car should be about five by six feet inside. For outside dimensions, allow six inches

all around except in back, where space for hoisting machinery must be provided. Safety devices, acting on the governor principle, should be furnished to prevent excessive speed; an air cushion should be installed at the bottom of the pit.

The car and its safety devices are the same for electric elevators. Here it is operated either by a drum carrying a hoisting cable or a screw with a traveling nut connected with a system of multiplying sheaves and cables. The counter-weight is made equal to the weight of the car plus one-half the maximum live load.

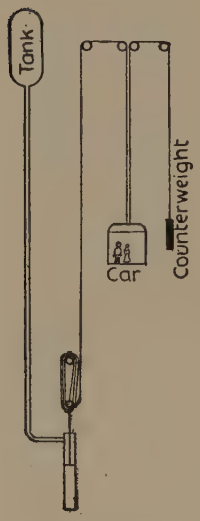

FIG. 145*b*.—Hydraulic Elevator.

The horse-power necessary to operate a hydraulic elevator is equal to the maximum unbalanced weight in pounds multiplied by speed in feet per minute divided by 33,000. The elevators going up are in motion only about one-third of the time and hence the average theoretic horse-power will be one-sixth of the maximum. To provide for emergencies and friction, furnish about one-half the power required to lift all elevators with maximum load at maximum speed.

In electric elevators, it is necessary to overcome only friction and unbalanced weight. Very roughly, this will require 20 horse-power per car upon an average. It may, however, be increased to 30 at starting.

The hydraulic elevator is preferred in practice, as the greater economy of the electric type is offset by the large fluctuations in power required in starting.

The freight elevators are much like the passenger type except that they are designed for heavier loads and slower speeds. Ash hoists, small elevators of the chain or plunger type are usually provided in the basement.

(*8*) *Ventilation*

This is commonly furnished to toilets, first floor, and basements. It is usually assumed that other rooms are self-ventilating. A model building, however, should be completely

ventilated. Not less than 2000 cubic feet per person, nor less than four times the cubical contents of the room should be removed each hour. City air must in general be purified, either by cheesecloth filters or by introducing a washing apparatus consisting of sheets or sprays of water. The pressure will seldom exceed 3 ounces per sq.in. and the velocities may be taken as about 40 feet per second in the mains, 15 in the branches and 2 to 5 at the outlet. A similar system is advisable to exhaust the air from the rooms. Disk or centrifugal fans may be used for this purpose. The removal of 2000 cubic feet per minute will require 0.7 horse-power at the fan and one at the engine.

(9) *Heat*

Direct radiation is used almost exclusively. The heating agent is exhaust steam from the power plant, provision being made for the admission of live steam from the boiler in case the other is not sufficient. Radiating surface required may be obtained by consulting books on the subject. As a very rough rule for the latitude of New York, we will need 1 to 3 sq.ft. for every 100 cu.ft. of space to be heated. This amount will vary with the proportion of window space and the exposure of the walls. The radiators are commonly placed under the windows. The diameter of the pipe in inches may be obtained approximately for a return system by dividing square root of radiating surface in feet by 10. To prevent back pressure on the engines, a return is obtained by exhausting from the engine room.

(10) *Light*

Incandescent lamps in 16 candle-power units are typical of office buildings. The necessary candle-power equals the floor area in sq.ft. divided by 3 or 4. This will take 1 watt (.0013 horse-power) per sq.ft. at the lamp terminals. Allowing 75% efficiency for dynamos and so forth, and 75% of total installation in use, this means .0013 horse-power required per sq.ft. for lighting purposes. If arc lamps are used, increase above values by 40%. Both the two- and the three-wire systems are employed. For the former 120 volts pressure is maintained;

for the latter, 230; both are direct current. This is furnished by generators direct connected to their engines. Machines are in several units, often of different capactities to obtain economy of operation under partial load. In laying out a plant the following points should be borne in mind:

(*a*) The possibility of nearly all the lights being turned on during dark days.

(*b*) This maximum demand might occur when one of the machines was disabled.

(*c*) Machines are often designed so that they may be temporarily overloaded with but small loss of efficiency.

(*d*) An emergency connection can sometimes be made with an outside supply.

(11) *Telephone and Telegraph*

Ducts for these purposes should be placed in those rooms where they are likely to be required. Provision for mail chutes, electric bells, buzzers, and for messenger service should not be forgotten.

Steam Plant

For furnishing the necessary steam, water-tube boilers are preferred to fire-tube in spite of their greater cost, because with the former, the effects of an explosion are confined to the immediate vicinity. The chimney is made of riveted steel pipe, about $\frac{1}{4}''$ thick, and is lined with fire brick all or a part of the way. It is located in space least desirable for renting purposes and extends to the roof. Within the limits usual in office buildings, its size may be determined approximately by the following formula:

$$H = 1.6\frac{2d-1}{d}A\sqrt{h};$$

where H is the horse-power;

d, the inside diameter of the chimney in feet;

h, the height of the chimney in feet;

A, the clear area of the chimney in sq.ft.

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Engineering Record, Vol. 54, p. 614.
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Elevators. "Hydraulic Elevators," by Baxter.
"Elevators Hydraulic and Electric," by Swingle.
"General Electric Review," Vol. 16, p. 474.
"Vol. 5, Cyelopedia of Architecture, Carpentry, and Building," by American Technical Society.
Electrical World, Vol. 62, p. 125.
Journal of the Association of Engineering Societies, Vol. 51, p. 208.
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Heating and Ventilation. "Heating and Ventilation," by Hoffman.
"Steam Heating and Ventilation," by Monroe.
"Elements of Heating and Ventilation," by Greene.
Engineering News, Vol. 41, p. 63.
Engineering Record, Vol. 66, p. 63.

Electric Lighting. "Electric Lighting," by Crocker. Vols. 1 and 2.
"Electric Lighting," by Franklin.
Electrical World, Vol. 62, p. 25.

Prob. 145. How many elevators containing 25 sq.ft. each are needed for building of Art. 147? Rated speed, 500 feet per minute.

Art. 146. Specifications

We find less stringent specifications here than for bridges. While there is no doubt that the failure of a high office building would be more disastrous, there is little vibration and the live load seldom reaches anything like the maximum. The usual practice is to specify for the latter from 60 to 150 pounds per square foot. Investigation of actual conditions in office buildings has shown that under average circumstances, this may vary from 10 to 40 pounds.* However, Johnson found that a selected crowd of people might weigh as high as 181 pounds per square foot.† In view of this, live load should not be reduced below 75 pounds and 100 seems advisable for the average office building. Where machinery is used to an appreciable extent, the floor should be designed for a live load of not less than 150 pounds and supports for especially heavy machines should be computed for their actual weight, increased by an allowance, varying with the speed and vibration, with a maximum equal to the original weight. Schneider advocates the consideration of an alternative concentrated load which he places at 5000 pounds.‡ In this case the live load may be reduced somewhat but not more than 20 pounds per square foot. The author prefers the uniform *plus* a concentrated load for the floor system.

For the roofs, the concentrated load may be omitted. Otherwise a flat roof should be designed for the same loads as for floors. If the public will not have access thereto, it may be computed for dead plus snow. Bear in mind, however, that an accident, alterations in the building, or some unusual spectacle may cause it to be crowded. Consider also the possibility of the fall of a portion of a neighboring building due to a fire or other accident. Pitch roofs are figured for the same loads as mill buildings.

The concentrated load should also be omitted for the columns. A further reduction is often made for posts below the top story. It hardly seems reasonable to suppose that the tributary area on all floors could be simultaneously loaded with 80 pounds

* See Am. Soc. Civil Engineers, Vol. 54, Part 7, p. 373.
† Ibid., p. 441.
‡ Ibid., p. 375.

per square foot. The area of the column footing on compressible soils should be designed for the dead plus probable live. However, no dangerous stresses should occur when dead, wind, and full live, reduced as noted for columns, are acting.

We will give points in which the specifications for the steel frame of an average office building differ from those in Art. 50 for railroad bridges.

Minimum requirements should not be less than those of the city building laws. The latter are often accepted in entirety.

Specifications for High Office Buildings

Omit (*a*) *Description*, Art. 50.

Rewrite (*b*), *Loads*, as follows:

(*1*) Dead Loads — Dead loads shall be checked in all cases from architect's and contractor's cross-sections and details.

(*2*) Live Load for Floors — The live load on first floor shall be taken as 100 pounds per square foot plus a concentration of 5000 pounds anywhere. Use same values for remaining floors except that 80 is substituted for the uniform load.

(*3*) Live Load for Flat Roofs — The live load on a flat roof shall be taken as 80 pounds per square foot.

(*4*) Live Loads for Pitch Roofs — The live loads on pitch roofs are to have the same values as for Mill Buildings, Art. 131. For *C*, use 30.

(*5*) Live Loads for Columns — The live load on the columns consists of *uniform* loads on floors and roofs as specified above; this to be taken at its full value on column in story below; 95% of same, two stories below; 90%, three stories below; and so on. Maximum reduction, 50%.

(*6*) Wind Loads — The wind pressure on exterior walls shall be taken as 30 pounds per vertical square foot acting in any horizontal direction.

(7) LOADS FOR FOOTINGS

Footings shall be designed for dead loads plus 20 pounds per square foot for all floors. If dead plus live as reduced for columns plus wind exceeds twice the allowable bearing values as given below, *all* areas must be proportionately increased.

(c) *Allowable Unit Stresses in lbs. per sq.in.*

For medium steel, allow as follows:

(1) MEDIUM STEEL

Tension on net section	16,000
Compression on gross section	

$$16{,}000 - 90\,\frac{\text{length in inches}}{\text{least radius of gyration in inches}}$$

Shear on shop rivets, pins, and gross sections of webs	12,000
Shear on field rivets and bolts	9,000
Bearing on shop rivets and pins	24,000
Bearing on field rivets and bolts	18,000
Bending on pins	24,000
Other flexural stresses	16,000

(2) MASONRY

Allowable compression on wall masonry:

Brickwork, lime mortar	100
Brickwork, Portland cement mortar	150
Concrete, 1 Portland cement, 3 sand, and 6 stone	250
Stone masonry, first class	250

These values may be increased 50% for local bearing on bulky masonry as in foundations.

Allowable values in compression direct or flexural, reinforced concrete:

1 Portland cement, 2 sand, 4 stone	500
1 Portland cement, 1 sand, 3 cinder	200

Footing areas shall be designed for the following values:

(3) FOUNDATIONS	On soft clay	15
	Wet sand	15
	Ordinary clay	30
	Dry clay	45
	Dry sand	50
	Coarse sand	60
	Gravel	75
	Hardpan, (cemented gravel)	150

(4) COMBINATIONS Above values are for dead plus live; for combinations including wind, allowable stresses may be increased 25%.*

Most of the remainder of Art. 50 may be used without alteration. We will give a few changes which are usually made.

(*d* 1) Accessibility of details may be omitted.

In (*d* 2), metal $\frac{5}{16}''$ thick is allowable.

Substitute for (*d* 5), which penalizes angles connected by one leg, the following:

DEFLECTION LIMIT Beams will possess sufficient strength to carry their loads including the dead weight of walls without any assistance from the masonry. So loaded deflections shall not exceed $\frac{1}{360}$ of the span.

Omit (*d* 8) concerning counters.

In (*d* 9), maximum slenderness ratios become 150 and 125 for bracing and other members respectively.

Omit (*d* 15).

(*e* 5) for splices shall be changed to read:

"Column ends shall be milled and the splices have sufficient strength to render the shaft fully continuous."

In (*f* 2), material instead of being sub-punched and reamed, is punched $\frac{1}{16}''$ larger than diameter of the rivet.

*Except for footings, which will be governed by *b* 7 above.

In (f 7) and (f 9), clauses forbidding welded and adjustable members are more or less of a dead letter.

REFERENCE

American Society of Civil Engineers, Vol. 54, Part VII, p. 371.

Prob. 146. How much steel will be saved for each floor by the omission of the concentrated load in the next article?

Art. 147. Computations for a High Office Building

Let us design and detail portions of the building, a section of which is shown in Fig. 134d. Suppose it to be 10 stories high, the first 20, and the others 14 feet; the basement is also taken as 20 feet. Let us provide for a live load of 100 lbs. per sq.ft. on the first floor and 80 lbs. on those above and the roof. Except the latter the beams will be designed for a single additional concentrated load of 5000 lbs. placed anywhere. The wind pressure is taken at 30 lbs. per sq.ft. and is carried by bracing, which is shown together with the columns in Fig. 147a. We will employ allowable stresses of preceding article.

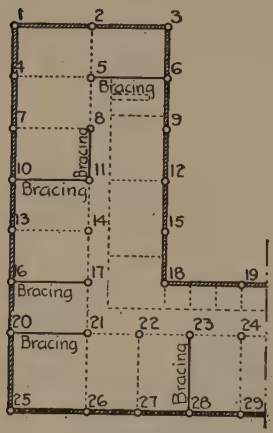

Fig. 147a.
Arrangement of Bracing.

The wall 14 feet high and 12″ thick will be considered to weigh 1500 lbs. per lin.ft. For the floor, we will use a 12″ flat terra cotta arch weighing 44 lbs. per sq.ft., Art. 138. To this we must add 20 lbs. for cinder concrete filling, 8 lbs. for wooden flooring, 5 lbs. for plaster and 20 lbs. for partitions, making a total of 97 lbs., say 100 lbs. per sq.ft. or 600 lbs. per lin.ft. per stringer when they are spaced 6 feet apart.

We will now tabulate the results of computations for most

of the beams which occur in a typical upper floor of this building. There are other cases, but this will illustrate the method, not only for all the floors, but for the roof, where the dead load may be somewhat lighter. A number of approximations will be noticed. For example, the concentration of 5000 lbs. represents a safe which could not bring its entire load on a wall beam. It will be seen that it is sometimes more economical to use a shallow beam and pay the additional cost of the steel rather than to increase the height of the building. Shear determines the size of the beam only when it is short and carries heavy loads; however, it should always be tested.

	Stringers.				Girders.	
	Wall		Floor.		Wall.	Floor.
Span in feet	12	18	12	18	18	18
Loads in Lbs.						
Wall per lin.ft. unif.	1,500	1,500	.....	.....	1,500	
Dead, floor ft. unif.	300	300	600	600		
Dead, floor concentrated	.....	.....	.....	.....	3,800	7,600
Dead, beams, per lin. ft.	65	90	40	65	100	70
Live, per lin. ft. uniform	240	240	480	480		
Live, concentrated	.....	.....	.....	.....	2,880	5,760
Concentrated loads	5,000	5,000	5,000	5,000	5,000	5,000
Shears in lbs.						
Wall	9,000	13,500	.....	.....	13,500	
Floor	1,800	2,700	3,600	5,400	3,800	7,600
Beam	390	810	240	580	900	630
Live uniform	1,440	2,160	2,880	4,320	2,880	5,760
Concen. load	5,000	5,000	5,000	5,000	3,330	3,330
Total	17,630	24,170	11,720	15,300	24,410	17,320
Moments in Ft.-lbs.					60,750	
Wall	27,000	60,750	.....	.....	22,800	45,600
Floor	5,400	12,150	10,800	24,300	4,050	2,830
Beam	1,170	3,650	720	2,630	17,280	34,560
Live uniform	4,320	9,720	8,640	19,440	20,000	20,000
Live concentrated	15,000	22,500	15,000	22,500		
Total	52,890	108,770	35,160	68,870	124,880	102,990
Total in kip ins.	635	1,305	422	827	1,499	1,236
Total divided by 16.0 (*S*), Section modulus required	39.7	81.6	26.4	51.7	93.7	77.3
Use	2 I's	2 I's	1 I	1 I	2 I's	1 I
	10″ @ 25	15″ @ 42	12″ @ 31.5	15″ @ 42	15″ @ 42	15″ @ 60
Section modulus furnished	48.8	117.8	36.0	58.9	117.8	81.2
Gross area for shear, sq.in.	6.20	12.30	4.20	6.15	12.30	8.85
Shear in unit stress, lbs. per sq.in.	2840	2000	2790	2480	1980	1960
which allow ample margin for rivet holes.						

Tie Rods

The tie rods are inserted in accordance with the usual practice. Let us test them for a live load of 150 lbs. per sq.ft. representing the given live load plus an allowance for the addi-

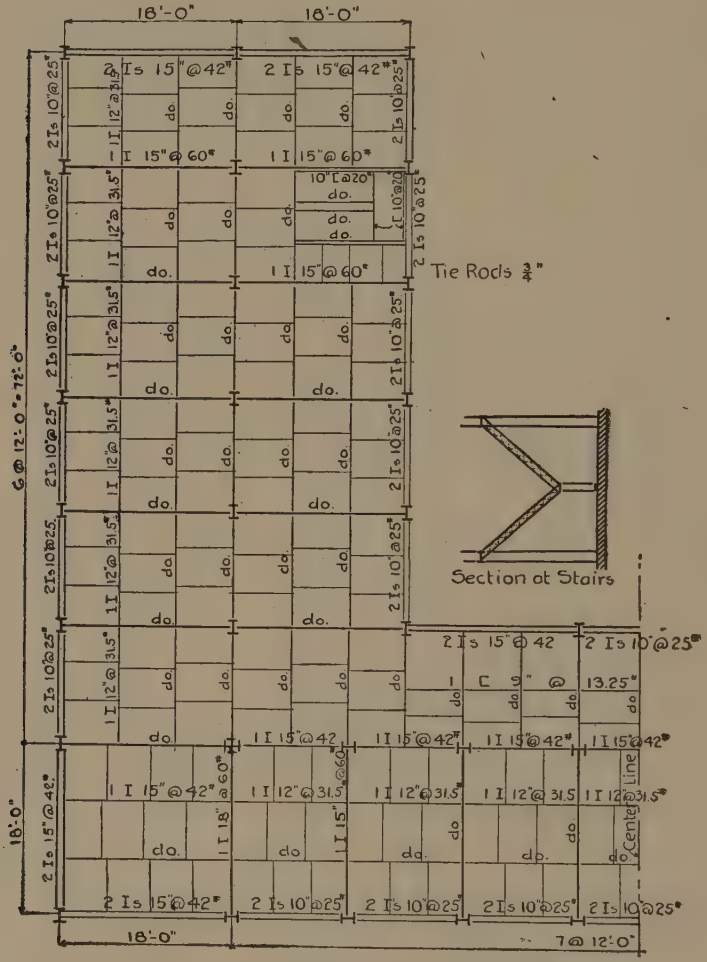

Fig. 147*b*.—Framing Plan for an Upper Floor.

Rivets ¾". Open Holes $\frac{13}{16}$".

tional stress caused by the concentration. From Art. 138, the rise will be 9.6″. From Art. 137,

$$H = \frac{wL^2}{8R} = \frac{150 \cdot 6 \cdot 6}{8 \cdot 0.8} = 844 \text{ lbs. per lin.ft.}$$

The area of a $\frac{3}{4}$″ rod at the root of the thread is 0.302 sq.in. With a maximum spacing of 4.5 feet the tensile unit stress is:

$$844 \cdot 4.5/.302 = 12{,}600 \text{ lbs. per sq.in.}$$

The flexural stress due to the thrust in the 12″ I @31.5#, 4 feet between supports, is,

$$\frac{WLc}{8I} = \frac{844 \cdot 4 \cdot 48 \cdot 2.5}{8 \cdot 9.50} = 5330 \text{ lbs. per sq.in.}$$

For a 15″ I @42#, 4.5 feet between supports,

$$= \frac{844 \cdot 4.5 \cdot 54 \cdot 2.75}{8 \cdot 14.62} = 4800 \text{ lbs. per sq.in.}$$

Either of which is safe according to the rule developed in Art. 137.

Fig. 147*b* represents typical framing plan for the upper floors. In the lower stories, some modification may be necessary in the panels containing bracing. Fig. 147*c* shows the detail of a representative floor stringer; Fig. 147*d*, that for a floor girder; Fig. 147*e*, that for a wall stringer.

Wind Bracing

To exemplify method let it be required to design wind bracing between columns 20 and 21, Fig. 147*a*. This set may be assumed to carry $24/90 \times \frac{1}{2} = 2/15$ of wind. The panel load for the 14-foot height is hence:

$30 \cdot 14 \cdot 92 \cdot 2/15 = 5150$ lbs. At the top we will use a little more than a half panel load. At first and second floors, loads will vary as panel lengths. Loads and stresses in the table below are in kips.

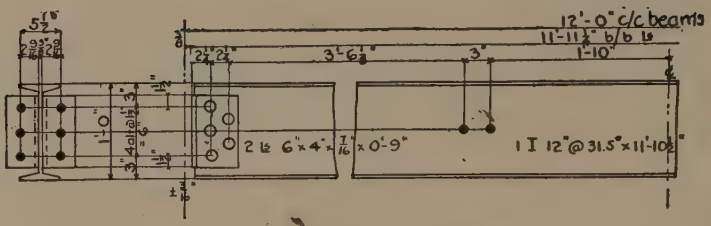

FIG. 147*c*.—Typical Floor Stringer.

(A majority of the floor stringers in the building will be exactly like this.)

Rivets ¾". Open Holes 13/16".

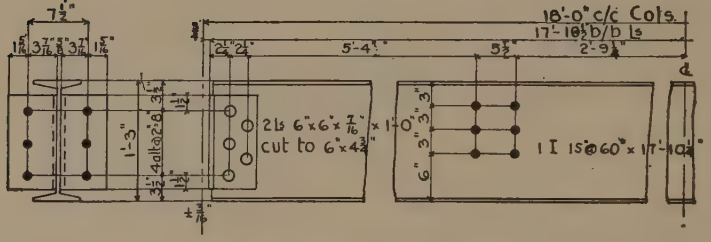

FIG. 147*d*.—Typical Floor Girder.

(18-foot floor girders where there is neither wall nor wind bracing will be like the above except for variations caused by change of size of column.)

Rivets ¾" Open Holes 13/16".

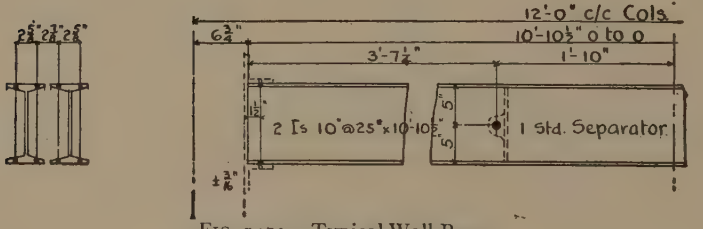

FIG. 147*e*.—Typical Wall Beam.

(12-foot wall beams will be like this above the first floor except for change in size of columns.)

Rivets ¾". Open Holes 13/16".

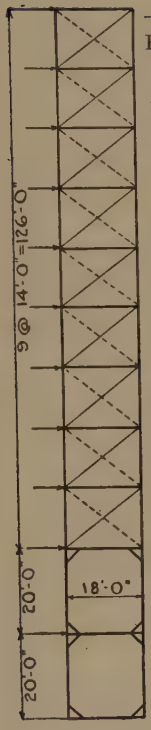

FIG. 147*f*.

Floor.	Stresses. Load.	Struct. C	Columns. Windward. T	Columns. Leeward. C	Columns. Diagonal. T	Diagonals. Use 1 L.	Area sq. in. Gross.	Area sq. in. Net.	Unit Stress.	$\frac{3}{4}$" Field Rivets.
Roof	3.00	3.0	0	2.3	3.8	3"×3"×$\frac{5}{16}$"	1.78	1.51	2.5	2
10th	5.15	8.1	2.3	8.6	10.3	do.	do.	do.	6.8	3
9th	5.15	13.3	8.6	19.0	16.9	do.	do.	do.	11.2	4
8th	5.15	18.5	19.0	33.4	23.4	do.	do.	do.	15.5	5
7th	5.15	23.6	33.4	51.8	30.0	do.	do.	do.	19.9	6
6th	5.15	28.8	51.8	74.2	36.4	3"×3"×$\frac{3}{8}$"	2.11	1.78	20.3	8
5th	5.15	33.9	74.2	100.6	43.0	3$\frac{1}{2}$"×3$\frac{1}{2}$"×$\frac{3}{8}$"	2.48	2.15	20.0	9
4th	5.15	39.0	100.6	131.0	49.5	3$\frac{1}{2}$"×3$\frac{1}{2}$"×$\frac{7}{16}$"	2.87	2.48	20.0	10
3rd	5.15	44.2	131.0	165.4	56.0	3$\frac{1}{2}$"×3$\frac{1}{2}$"×$\frac{1}{2}$"	3.25	2.81	19.9	12
2nd	6.25	25.2	193.4	193.4						
1st	3.70	1.85	251.5	251.5		Allowable			20.0	5.0
										in single shear
Basement		—				Max. moment = $\frac{1}{2}$ Pl. = $\frac{1}{2}$ 25.2·20 = 252 kip ft.				
						Max. moment = $\frac{1}{2}$ 27.0·20 = 270 kip ft.				

DETAILS OF WIND DIAGONALS

Below is the detail of the laterals in the fourth story between columns 20 and 21. Others are usually made as much like them as possible to economize on templet work, the rivets being omitted and sizes decreased as we go up. For less than five rivets, lug angles may be omitted.

Design of the Wind Struts

As beams, the struts of Fig. 147*f* also carry a dead uniform load of 350 lbs. per lin.ft.; two dead concentrated loads of 4000 lbs. each at the third points; a live uniform load of 240 lbs. per lin.ft.; two concentrated live loads of 2900 lbs. at third points; and a concentrated live load at the middle of 5000 lbs. The moment at the middle of the beam due to these forces is:

$$590\times18\times18\times12/8+6900\times6\times12+5000\times18\times12/4=$$
$$1{,}053{,}500 \text{ in.lbs.}$$

The resulting fiber stress is for a 15″ I @ 60# .

$$S=Mc/I=1{,}053{,}500/81.2=12{,}800 \text{ lbs. per sq.in.}$$

According to the framing plan, Fig. 147*b*, this beam is a 15″ I @ 60# . It is held along its entire length and especially

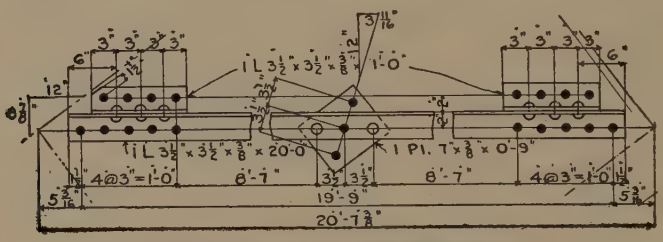

Fig. 147*g*.—Detail of a Wind Diagonal.
Rivets ¾″. Open Holes $\frac{13}{16}$″.

at every third point. Taking its unsupported length as 6 feet, the allowable unit compression is,

$$S=1.25(16{,}000-90l/\rho)=13{,}300 \text{ lbs. per sq.in.}$$

Total allowable compression is:

$$17.67(13{,}300-12{,}800)=8800 \text{ lbs.}$$

The two upper struts may hence be made 15″ @ 60# . In a similar way, it may be computed that 15″ @ 65# will do at 8th and 9th floors; 15″ @ 70# at 6th and 7th; 15″ @ 75# at 3d, 4th, and 5th. Below the third floor, the bending moment due to wind loads as given above will not cause any increase of size. For the moment due to the wind is a max-

imum where that due to vertical loads is zero and vice versa. Hence figuring for compression and moment of the latter, we use an I, 15″ @ 70# at 2d floor and @ 60# at 1st and basement.

Columns

As an example, let us compute column for # 21, using a built I section, $12\frac{1}{2}$″ b. to b. L's, longer legs outstanding.

Floor.	Loads in Kips. Dead.	Live.	Wind.	Column.		Total.	Use.	Area, Sq.in.	Rad. of Gyr. Inches.	Unit Stresses. Allow.	Actual.
Roof	25.0	18.0		1.0							
			2.3		Without Wind	44.0	4 L's 6″×4″×$\frac{3}{8}$″ latticed	14.44	3.00	11.0	3.1
10	25.0	17.1		1.0							
			8.6			87.1					6.0
9	25.0	16.2		1.0							
			19.0			129.3	4 L's 6″×4″×$\frac{7}{16}$″ latticed	16.72	3.00	11.0	7.7
8	25.0	15.3		1.0							
			33.4			170.6					10.2
7	25.0	14.4		1.5							
			51.8		With Wind	263.8	4 L's 6″×4″×$\frac{9}{16}$″	27.24	2.61	12.7	9.7
6	25.0	13.5		1.5			1 Pl. 12″×$\frac{1}{2}$″				
			74.2			326.2					12.0
5	25.0	12.6		2.0							
			100.6			392.2	4 L's 6″×4″×$\frac{9}{16}$″				10.0
4	25.0	11.7		2.0			1 Pl. 12″×$\frac{1}{2}$″	39.49	3.12	13.9	
			131.0			461.3					11.7
3	25.0	10.8		2.0			2 Pls 14″×$\frac{7}{16}$″				
			165.4			533.5					13.5
2	25.0	9.9		4.0							
			193.4			600.4					
1	25.0	13.5		4.0							
			251.5			701.0	See Below.				
Base.	25.0	13.3									
			281.5			769.3					

The dead weight is that previously computed for the floor plus the weight of the beams. Live is the uniform without the concentrated, reduced as required in the specifications. Wind stresses are taken from the computations for the diagonals above. Column loads are a rough approximation estimated in advance. Details of sample columns are given in Figs. 147*j* and 147*k*.

We will next prepare a table for the design of the columns in the two lower stories which are subjected to a combined compression and bending. Dimensions are in inches.

Piece.	Area.	I_{YY}.	I_{XX}.	Min. Radius of Gyr.	c_{XX}.	Section Modulus XX.
(1) 1 Pl. 12″×½″	6.00	000	72			
(2) 4 L's, 6″×4″×$\frac{9}{16}$″	21.24	186	610			
(1) and (2)	27.24	186	682	2.61		
(3) 2 Pls. 14″×$\frac{7}{16}$″	12.25	200	513			
(1), (2), (3)	39.49	386	1195	3.12		
(4) 2 [′s, 15″ @ 45#	26.48	750	980			
(1), (2), (4)	53.72	936	1662	4.17	6.87	242
(5) 2 Pls. 15″×⅜″	11.25	211	560			
(1), (2), (4), (5)	64.97	1147	2222	4.20	7.25	306
(6) 2 Pls. 15″×⅜″	11.25	211	621			
(1), (2), (4), (5), (6)	76.22	1358	2843	4.22	7.62	373
(7) 2 Pls. 15″×⅜″	11.25	211	686			
(1), (2), (4), (5), (6), (7)	87.47	1569	3529	4.24	8.00	441
(8) 2 Pls. 15″×⅜″	11.25	211	754			
(1), (2), (4), (5), (6), (7), (8)	98.72	1780	4283	4.25	8.37	510

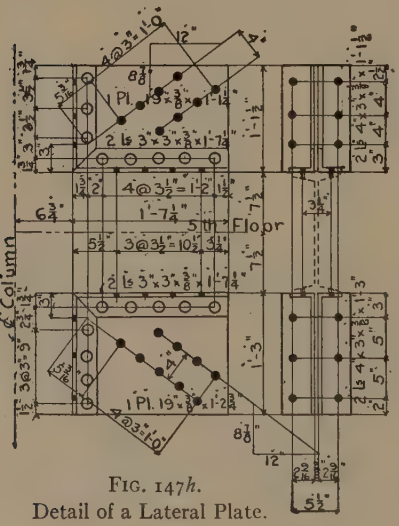

FIG. 147*h*.
Detail of a Lateral Plate.

Column, first to second floor. Length, 20 feet. Compression with wind, 600.4 kips. Maximum moment, 3024 kip in. All quantities in kips and inches.

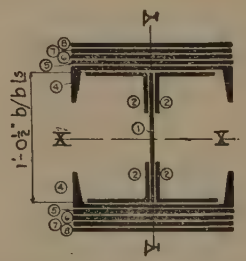

FIG. 147*i*.—Column Section.

Pieces.	Unit Stresses.			Allow. Moment.	Cut off from Center.
	Allow.	Actual Comp.	Allow Flex.		
(1), (2), (4)	13.5	11.2	2.3	557	
(1), (2), (4), (5)	13.6	9.3	4.3	1318	22
(1), (2), (4), (5), (6)	13.6	7.9	5.7	2130	52
(1), (2), (4), (5), (6), (7)	13.6	6.9	6.7	2960	84
(1), (2), (4), (5), (6), (7), (8)	13.7	6.1	7.6	3880	116

Column, basement to first floor, Length, 20 feet, Compression with wind, 701.0 kips. Maximum moment, 3240 kip in.

(1), (2), (4)	13.5	13.0	0.5	121	
(1), (2), (4), (5)	13.6	10.9	2.7	828	
(1), (2), (4), (5), (6)	13.6	9.3	4.3	1605	30
(1), (2), (4), (5), (6), (7)	13.6	8.1	5.5	2430	60
(1), (2), (4), (5), (6), (7), (8)	13.7	7.2	6.5	3320	90

These cut-off points are easily obtained; since the shear is constant, moment varies directly as the distance from the center. Notice that a plate is cut off at the point where the full strength of the plate underneath is necessary. Plate (8) need not be used in the upper column at all, as the knee brace will carry the additional moment.

Cantilever Foundation Girder

Next let us design the cantilever girder for columns 20 and 21 which is shown in Fig. 147*m*. Dimensions are as given in Fig. 147*l*. The distance 2′ 10″ is made as small as possible, and still keep footing on the property. Neglecting own dead weight, which is relatively of small importance, the totals are in kips:

	Dead.	Live.	Wind.	Totals.	
				Wind R.	Wind L.
On Col. 20	475C	100C	282T or C	857C	293C
On Col. 21	321C	166C	282C or T	205C	769C

The preceeding table shows that the larger stresses in the girder occur with the wind right. Fig. 147*l* shows loads, shears, and moments for this case. It will be noted that there is a

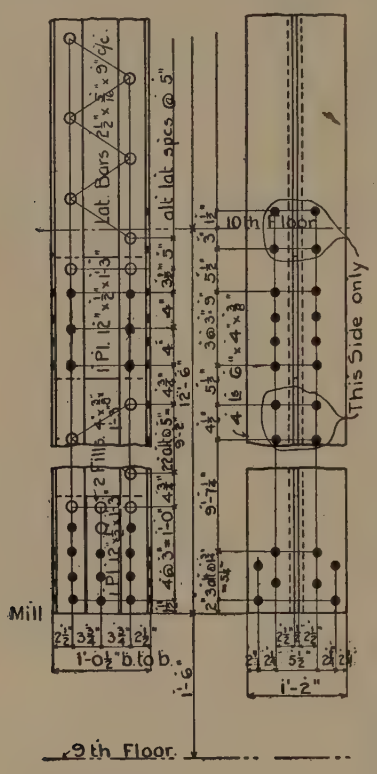

FIG. 147*j*.

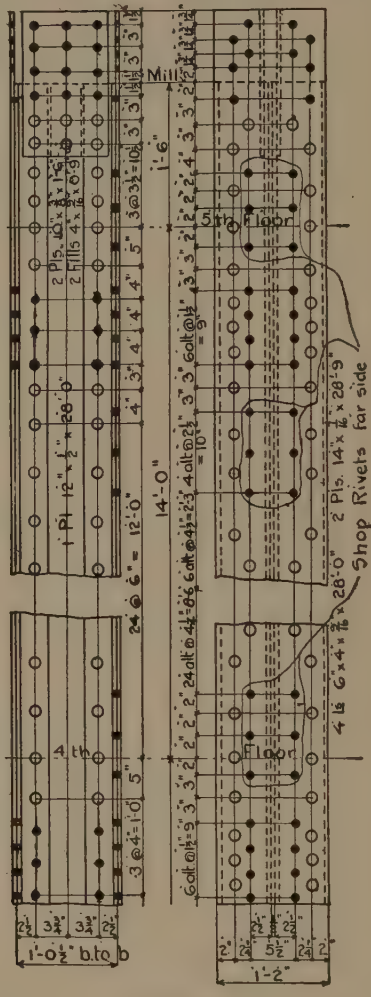

FIG. 147*k*.

Details of Column 21.

chance for an uplift. Hence column will have to be anchored or braced to those adjacent. We will make the depth small, say about three feet, in order to lessen excavation and to

give stability. A box girder with three webs, $7\frac{1}{2}''$ center to center will be used. This is barely sufficient for clearance. In the shop, each web with its flange angles and stiffeners is first riveted up, next the center web is attached to the cover plates, and finally the side webs to the cover plates.

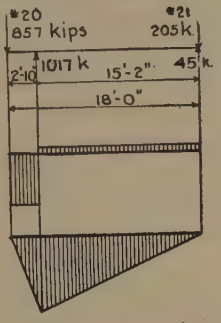

FIG. 147*l*.—Loads, Shears, and Moments for Cantilever Girder.

We will design with three webs, each $36''\times\frac{3}{8}''$, affording a gross area of 40.5 sq.in. At 15,000 lbs. per sq.in., this is good for 607 kips in shear.* On the left, this is exceeded and 857/15 equals 57.1 sq.in. is needed. We will use 2 plates, each $24''\times\frac{3}{8}''$ on the inside web, making a total gross area of 58.5 sq.in.

The maximum moment is $857\times34=29{,}100$ kip in. To the webs as already determined we will add 6″ angles and then enough cover plates to make the girder of sufficient strength. Computing by the exact method we may now tabulate, everything in inches:

	A. Areas.		Distance to c. g.	*Q* Stat. Moment	*I* Moment Inertia.
	Gross.	Net.*			
(1) 3 Web Pls. $36''\times\frac{3}{8}''$	40.5	30.4	....	...	3280
(2) 2 Web Pls. $24''\times\frac{3}{8}''$	18.0	13.5	....	...	650
(3) 8 Flange L's, $6''\times6''\times\frac{3}{8}''$	34.9	29.6	16.6	492	8240
(4) 2 Cover Pls. $30''\times\frac{7}{16}''$	26.2	23.3	18.5	431	7980
(5) 2 Cover Pls. $30''\times\frac{7}{16}''$	26.2	23.3	18.9	440	8320

* Deduct for $\frac{1}{4}''$ rivets, 2 in each angle and 4 in cover plates.

Various possible combinations have the following properties:

	I	*c*	*I*/*c*.	Allowable Moment. $S=20$ kips per sq.in.
(1), (2), (3), (4)	20,150	18.69	1078	21,560
(1), (2), (3), (4), (5)	28,470	19.12	1489	29,780
(1), (3), (4), (5)	27,820	19.12	1455	29,110
(1), (3), (4)	19,500	18.69	1043	20,860

* Forces with wind call for more material in spite of larger allowable stresses.

Beginning at the left end of the cantilever, (1), (2), (3), (4), will do until moment becomes 21,560 kip in. at a point distant from the left of $21{,}560/857 = 25''$. At the right, (1), (3), (4), will do until moment reaches 20,860 kip in. at a point distant from the right of $20{,}860/160 = 130.''$

The $\frac{3}{4}''$ horizontal rivets will be three in a line, one in bearing on $\frac{3}{8}''$, and two in single shear, having a total value of 21.7 kips. The vertical rivets will be four in a line in single shear and possess a joint value of 26.5 kips. On the left:

(1), (2), (3), (4),

Spacing for horizontal rivets

$$= \frac{21.7I}{VQ} = \frac{21.7 \cdot 20{,}150}{857 \cdot 461} = 1.11''$$

Spacing for vertical rivets

$$= \frac{26.5I}{VQ} = \frac{26.5 \cdot 20{,}150}{857 \cdot 215} = 2.90''$$

(1), (2), (3), (4), (5),

Spacing for horizontal rivets

$$= \frac{21.7I}{VQ} = \frac{21.7 \cdot 28{,}470}{857 \cdot 681} = 1.06''$$

Spacing for vertical rivets

$$= \frac{26.5I}{VQ} = \frac{26.5 \cdot 28{,}470}{857 \cdot 435} = 2.02''$$

The spacing comes out too low; but we shall use the minimum $1\frac{1}{8}''$, as we have assumed concentrations instead of the distributed load seen in the details. On the right the shear is only one-fifth as much, hence spacing will be the maximum allowable.

STIFFENERS

Intermediate stiffeners will be $5'' \times 3'' \times \frac{5}{16}''$. At the reactions and loads they must be computed. Everything in kips and inches.

	Load.	Allow. Stress.	Area Req'd.	Use.	Area Furnished.
Left.....	857	20.0	42.8	16 L's, $5'' \times 3'' \times \frac{3}{8}''$	45.8
Center.....	1017	20.0	50.1	18 L's, $5'' \times 3'' \times \frac{3}{8}''$	51.5
Right.....	769	20.0	38.5	16 L's, $5'' \times 3'' \times \frac{3}{8}''$	45.8

Footings

Let it be required to design footings so that bearing pressure for dead plus probable live does not exceed 8 kips per sq.ft. This assumes a capacity to resist a somewhat higher pressure for a short time. We will now take loads and reactions as shown in Fig. 147*n*. The bearing areas required will be 75 and 34 sq.ft. For the larger we will make the area 7.5×10 feet, the former dimension being limited by the property line. The upper course of beams will have a maximum moment of $296.5 \times 30 = 8900$. kip in.

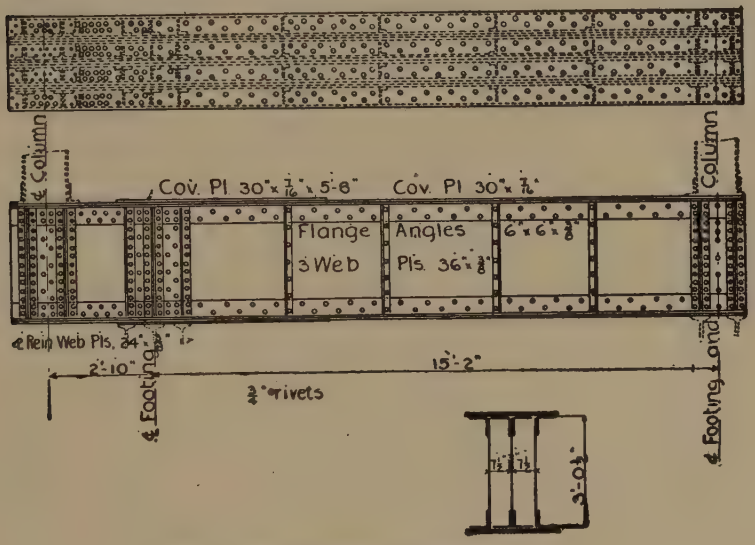

Fig. 147*m*.—Cantilever Foundation Girder.

Section modulus required is 8900/16 equals 556 in.^{3} We will use 3 I's, 24″@ 90#, spaced 9″ on centers. The maximum shearing unit stress is $296.5/3 \times 24 \times .63$ equals 6.6 kips per sq.in., O. K. For the lower set, the moment is 296.5×22.5 equals 6680 kip in. There will be needed a section modulus of 6680/16 equals 418. We will use 12 I's, 12″×31.5# with combined section modulus of 432 and a shearing resistance of $12 \times 12 \times 0.35 \times 12$ equals 605 kips. The other foundation may be treated in a like manner.

Truss

As an example, let it be required to omit columns 23 and 24, Fig. 147*a*, on the first floor. We will use a five-panel Pratt truss of the same depth as the floors, 14′ 0″, resting on column 22 on either side of the center. An outline of the truss is seen in Fig. 147*p*. These chords have in addition to their direct stresses, bending moments due to a dead uniform load of 560 lbs. per lin.ft., a live uniform of 240 lbs. per lin.ft., and concentrated loads at center of dead 4000 and live 8000 lbs. Maximum moment is 605 kip in., which will be neutralized by placing center of gravity lines of these members a counterbalancing distance above intersection points.

We will next determine areas and radii of gyration for mem-

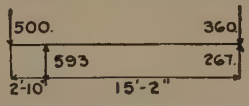

Fig. 147*n*.—Loads on Footing.

bers built up to provide for the stresses in this truss. See Fig. 147*q*. Everything in inches.

Member.	Area.	Distance to c. g. *xx*.	Statical Moment.	I_{xx}.	I_{yy}.
(1) 2 Pls. 18″×⅝″	22.5	0.00	0	607	635
(2) 4 L's, 4″×4″×⅝″	18.4	8.02	0	1210	895
(3) 1 Pl. 20″×⅝″	12.5	9.56	119.5	1142	417
(4) 2 Pls. 10″×⅝″	12.5	0	0	104	440
Totals	65.9	1.82	119.5	3063	2387

$$\text{Ic.g.} = 3063 - 65.9 \times 1.82^2 = 2847,$$

$$\text{Radius of gyration} = \sqrt{2387/65.9} = 6.00''.$$

(5) 2 Bars 4″×⅝″	5.0	9.56	47.8	456	298
Totals	70.9	1.01	71.7	3519	2685

$$\text{Ic.g.} = 3519 - 70.9 \times 1.01^2 = 3447,$$

$$\text{Radius of gyration} = \sqrt{2685/70.9} = 6.13''.$$

Strain Sheet for Truss

Loads at U_1 and U_2; Dead, 200; Live, 105. Loads at L_1 and L_2; Dead 25; Live, 15.
All values in kips and inches.

Member.	Stress.	Use.	Rad. of Gyr.	Areas.		Unit Stresses.	
				Gross.	Net.	Actual.	Allow.
U_1L_0	911C	Full Section.	6.13	70.9	....	12.8	12.8
U_1U_2, U_2U_3	888C	All but (5)	6.00	65.9	....	13.5	13.8
L_0L_1, L_1L_2	592T	2 Pls. 18″×$\frac{11}{16}$″, 4 L's 4″×4″×$\frac{5}{8}$″	....	43.1	37.0	16.0	16.0
L_2L_2	888T	All but (5)	....	65.9	55.5	16.0	16.0
U_1L_1	40T	4 L's, 3″×3″×$\frac{3}{8}$″	....	8.4	7.1	5.6	16.0
U_2L_2	335C	2 [’s, 15″@45 #	5.32	26.5	....	12.7	13.2
U_1L_2	487T	2 [’s, 15″ @ 33 #, 2 Pls, 15″×$\frac{1}{2}$″	....	34.8	30.4	16.0	16.0
U_2L_2	95T, 95C	2 [’s, 10″ @ 20 #	3.66	11.8	9.8	9.7, 8.0	16.0, 10.5

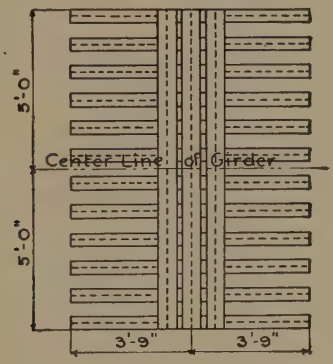

Fig. 147*o*.—Beams for Footing.

In some of the tension members above, the net area is made just sufficient to bring the actual stress down to that allowable, making it of course somewhat less than the gross. In detailing, care must be taken not to trespass on these limits.

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E. S. = Journal of Association of Engineering Societies.

Listed in Order of Number of Stories.

Periodical.	Volume.	Page.	Location.	Stories.	Erection.	Floor Plans.	Foundations.	Cantilevers.	Columns.	Beams.	Plate Girders.	Trusses.	Floors.	Roof.	Cornice.	Wind Bracing.	Walls.	Bay Windows.	Partitions.	Fireproofing.	Mech. Plant.
E. R.	63	591	Woolworth, N. Y.	55	*		*	*	*	*	*					*				*	
E. R.	65	177, 220																			
E. R.	66	97																			
E. R.	68	22																			
E. R.	55	146	Metropolitan Life, N. Y.	46			*		*	*					*	*					*
E. R.	55	116 et seq.	Singer, N. Y.	42			*	*	*	*	*				*	*					
E. N.	58	595	Singer, N. Y.	42			*		*	*						*					
E. R.	66	13	Seattle, Wash.	35			*	*	*	*	*		*	*		*	*				
E. R.	54	566, 603	City Invest., N. Y.	32			*	*	*	*	*			*		*					
E. N.	58	595	City Invest., N. Y.	32			*	*	*	*	*			*		*					
E. R.	63	152, 284, 476	Bankers Trust, New York	31			*	*	*	*	*	*				*					
E. R.	61	357	Bryant, N. Y.	30			*	*	*	*				*	*	*					
E. N.	36	226	Park Row, N. Y.	30		*	*		*	*			*			*			*		
E. R.	38	144	Ivins, N. Y.	29		*	*		*	*			*			*					*
E. R.	47	146	Belmont, N. Y.	26	*		*		*	*						*					
E. R.	65	354	McAlpine, N. Y.	25			*	*		*						*					
E. R.	64	508, 682	United Fire, N. Y.	25			*	*	*	*			*			*					
E. R.	61	320	Municipal, N. Y.	25			*		*	*	*	*		*	*	*	*		*		
E. R.	53	479	Wall St., N. Y.	25			*	*	*	*				*		*	*				
E. R.	46	242	New York	25	*		*		*	*				*		*					
E. N.	35	310	St. Paul, N.Y.	25		*	*		*	*						*					
E. R.	54	442 et seq.	Trust Co., N. Y.	25	*		*	*	*	*						*					
E. N.	48	217	Farmers Bank Pittsburgh	24		*	*		*	*					*	*					
E. R.	54	108	U. S. Exp., N. Y.	22	*		*		*	*					*	*					
E. R.	45	225	Hanover, N. Y.	22			*	*	*	*						*					
E. R.	48	388, 461	Land Title, Phila.	22			*	*	*	*	*				*	*	*				
E. R.	59	284	Masonic, N. Y.	21			*		*	*		*				*					
E. N.	70	976	Chicago, Ill.	21	*		*		*	*						*					
E. R.	56	121	Hudson Co., N. Y.	21			*		*	*						*	*				
E. R.	33	81	Amer. Surety	21	*		*	*	*	*						*					
	34	28	New York																		
E. R.	49	550 et seq.	Times, N. Y.	21	*		*		*	*	*		*	*	*	*	*				
E. R.	45	296	Flat Iron, N. Y.	21	*				*	*			*	*	*	*	*				
E. R.	56	515	Ger. Am., N. Y.	20			*		*	*						*					
E. R.	55	642	Tribune, N. Y.	20		*	*	*	*	*						*					
E. R.	49	19	Kuhn Loeb, N. Y.	20			*	*	*	*					*						
E. R.	45	34	Frick, Pitts.	20				*	*	*					*	*					
E. R.	35	140	Gillender, N. Y.	20			*		*	*				*		*		*			
E. R.	27	160 et seq.	Masonic Temple, Chicago	20		*	*		*	*	*			*							
E. R.	28	222 et seq.																			
E. N.	32	526	Am. Tract, N. Y.	20	*		*	*	*	*						*					
E. R.	55	418	Plaza Hotel, N.Y.	18					*	*	*		*	*							
E. R.	30	189	Manhattan Life, New York	18	*	*	*	*	*	*	*	*	*	*		*	*				*
E. R.	31	et seq. 5 et seq.																			
E. R.	47	338																			
E. R.	60	377	New York	17			*		*	*						*					
E. R.	58	581	San Francisco	17			*		*			*	*			*	*		*		*

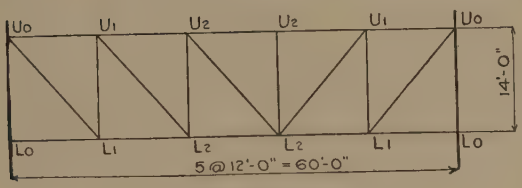


Fig. 147*p*.
Truss in Office Building.

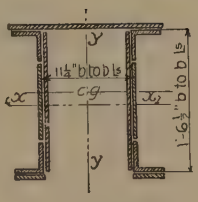


Fig. 147*q*.
Chord Section.

Periodical.	Volume.	Page.	Location.	Stories.	Erection.	Floor Plans.	Foundations.	Cantilevers.	Columns.	Beams.	Plate Girders.	Trusses.	Floors.	Roof.	Cornice.	Wind Bracing.	Walls.	Bay Windows.	Partitions.	Fireproofing.	Mech. Plant.
E. R.	49	484 et seq.	Imperial Hotel.....	17	*																
E. N.	30	486	Old Colony, Chi....	17																	
E. R.	50	206	New York.....	16	*																
E. R.	43	328 352	Mutual Life, N. Y. .	16		*	**		*	*	*		*								
E. N.	36	412	Astor Ho., N. Y....	16		*	*		*	*	*										
E. S.	26	125		16		*	*		*	*	*										
E. R.	37	412 433	Spreckels, San Francisco	15		*	*		*	*	*		*	*	*	*	*				*
E. R.	58	424	5th Ave., N. Y.....	14		*	*		*	*	*		*	*	*	*	*				*
E. R.	55	660	B. & O., Balt.....	13		*	*		*	*	*		*	*	*		*				*
E. R.	46	486 506	Hibernia, New Orleans	12		*	*	*	*	*	*		*	*	*		*				*

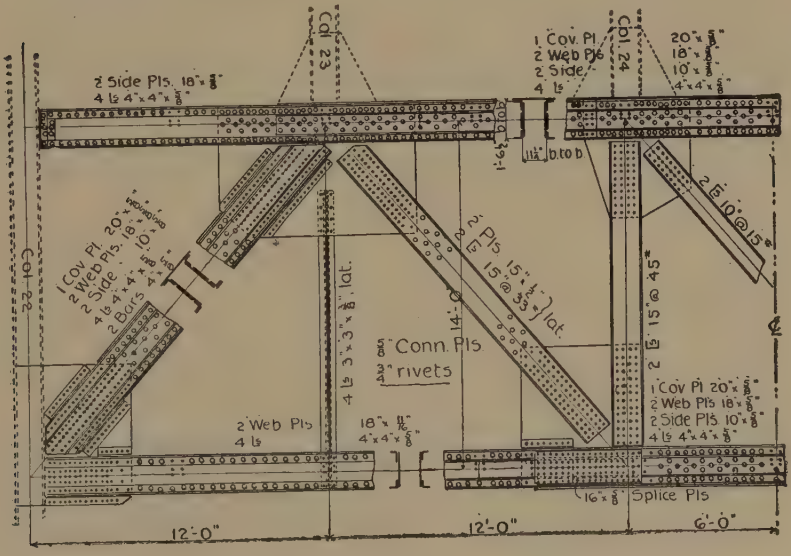


Fig. 147*r*.—General Drawing of Half Truss.

CHAPTER XII

MISCELLANEOUS BUILDING WORK

Art. 148. Steel for Brick Buildings

Most of these buildings will not be over four stories high. In general, they are of Type 3, Art. 123. For such heights, the brick walls can usually be relied upon to furnish the necessary resistance to wind pressure. Generally speaking exterior columns do not pay, as enough brickwork to carry the loads is required for stability. The heavy weights, more particularly those applied eccentrically or with vibration as in a crane, should be borne by steel columns. The lighter sections are common, the H or latticed channel being perhaps the best.

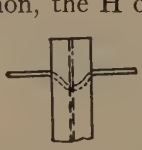


FIG. 148*a*.
Beam Anchor.

FIG. 148*b*.
Common Method of Computation.

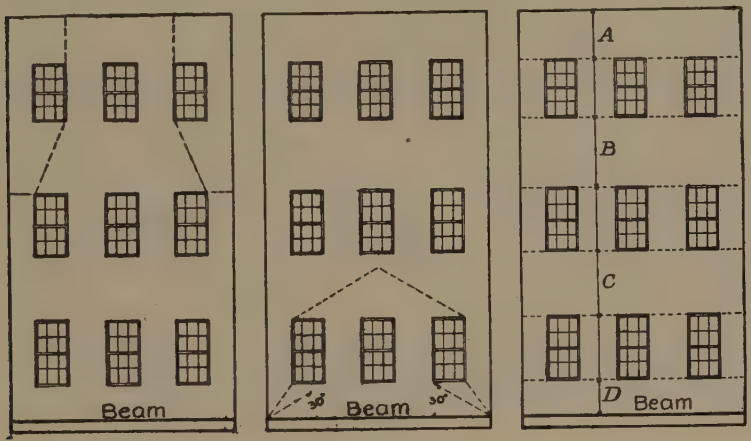
Beams are quite common but there is little new concerning them. Anchor firmly to brickwork, either by bolts through bottom flange or bearing plate or by a bent rod such as is shown in Fig. 148*a*. Always examine unit pressure on base and provide sole plate where necessary.

WEIGHT FROM BRICK WALLS

There is a large variation in practice where a beam carries a brick wall. The safest method undoubtedly is to neglect strength of latter as stated in Art. 142. However, we think, for solid brick walls, no beam need be considered to carry a height of brickwork greater than its span. Much depends on the quality of masonry, its age when load is first applied to the beam, and the nature of the building. Concerning the latter, a few cracks might not be objectionable in a workshop,

whereas they would be decidedly so in a church. Perhaps the most common method is to consider as carried by the steel all weight in an isosceles triangle whose base is the span and whose height is the half span times the tangent of 30°. See Fig. 148*b* for a graphical representation. If it is desired to consider strength, Baker recommends that brickwork be considered to be capable of carrying a load producing a flexural stress equal to tensile strength of mortar employed.

For broken walls, the most conservative method is shown in Fig. 148*c*, all loads and material below the dotted lines to be carried by the steel. Similarly, Fig. 148*d* shows applica-



Conservative. Common. Baker.

Methods for Broken Walls.

FIG. 148*c*. FIG. 148*d*. FIG. 148*e*.

tion of common method to this case. In Fig. 148*e*, Baker's method would figure capacity of strips *A*, *B*, *C*, and *D*, and subtract them from the total load. Beams must provide for the remainder.

Trusses are quite common for buildings having no other steel. However, they involve no important difference from those already mentioned in Art. 125. Sufficient bracing in the plane of the columns and trusses will be provided by a self-supporting wall. There should be bracing in the plane of the top chord, although roofing is often assumed to take

its place. Also note well that in curtain walls, bracing should usually be furnished, depending in a degree upon the nature of the loads and the exposure of the structure. As already mentioned, it may be omitted for low and comparatively broad buildings. The author believes that, save in cases where ample bracing is furnished by masonry of unquestioned rigidity and permanence, it should be furnished in all directions. *There is more trouble and more failures in building due to insufficient bracing than from any other structural fault.*

FIG. 148*f*.—Pittsburgh and Lake Erie R. R. Locomotive Shops. McClintic-Marshall Construction Co., Rankin, Pa.

Space does not allow us to go into the details of all the different buildings which come under this head. In general, they are quite simple and based upon principles already enunciated.

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Canadian Engineer, Vol. 19, p. 16. Engine House Terminals.
Electric Railway Journal, Vol. 32, 866. Car House Design.
Engineering News, Vol. 34, p. 23. Court House.
Engineer News, Vol. 34, p. 66. Railway Shops.

Engineering Record, Vol. 40, p. 690. Power Station.
Engineering Record, Vol. 44, p. 369. Church Roof.
Engineering Record, Vol. 51, p. 554. Power Station.
Engineering Record, Vol. 58, p. 646. Technical School.
Engineering Record, Vol. 59, p. 258. Soldiers' Memorial.

Prob. 148. Design, with an allowable flexural stress of 16,000 lbs. per sq.in., two I-beams of 15 feet span to carry a solid brick wall 20 feet high weighing 160 lbs. per sq.ft. Use each of the three methods given above.

Art. 149. Theaters

In a theater, we have a somewhat peculiar construction arranged to secure adequate sight and hearing for as many persons as possible in a given horizontal area. Similar methods are often employed for churches, music halls and so forth, but they will all be treated under the one head. The points to be particularly borne in mind are:

(1) *Capacity.*
(2) *Sight.*
(3) *Hearing.*
(4) *Exit.*
(5) *Fireproofing.*
(6) *Mechanical Conveniences.*
(7) *Mechanism for Producing Plays.*

We shall consider the latter as outside the scope of our work, except perhaps to indicate briefly the structural arrangements therefor.

(1) *Capacity*

This may be roughly estimated at one-eighth total inside area devoted to aisles and seats in square feet. Spacing of chairs back to back is 30 to 32 inches, and transversely 19 to 22 inches center to center. There should be aisles, perpendicular to the stage, 3 or 4 feet wide, spaced not more than 15 seats apart. Exterior passages in general equal or exceed 5 feet, safety in case of fire governing this distance. See (4)

(2) *Sight*

The height of the steps is fixed by the requirement that line of sight of each spectator shall be a certain amount, *d*, above the line of sight of spectator in front. A certain spot, *a*, Fig. 149*b*, on or perhaps below the stage, is selected as a desirable spot to be seen by all. A line from this point to 4′ 2″ above the floor at the back of any tier of seats is the line of sight for that tier. Seat is located about 2′ 8″ below. *d* is made 4 to 6″.

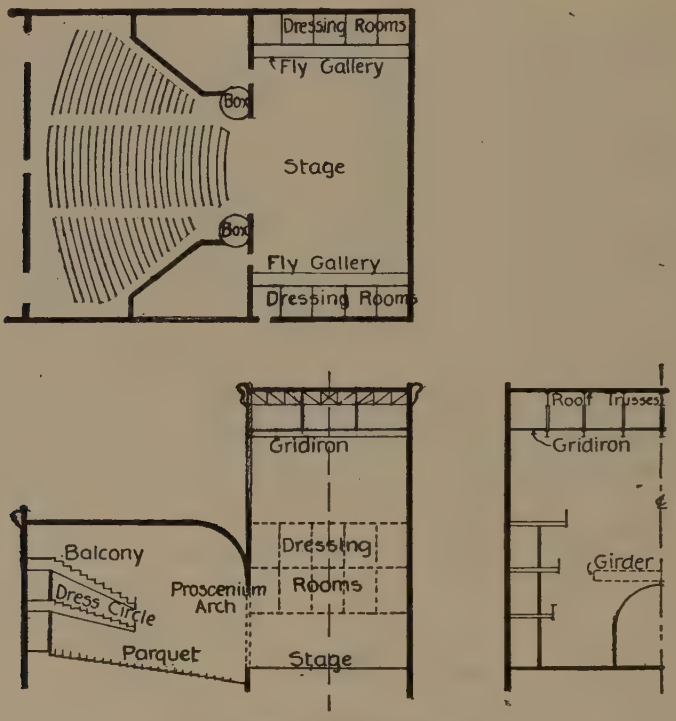

FIG. 149*a*.—A Typical Theater.

The chairs are usually arranged in arcs of circles, swung from a point on center line of theater, which is sometimes in front of, sometimes in back of the curtain line. For the upper floors, this center is much more likely to be in front of the curtain.

(3) *Hearing*

The maximum distance from curtain line to seat should not exceed 75 feet. Care should be taken to preserve acoustic properties as briefly mentioned in Art. 138.

(4) *Exit* *

City building laws govern here. The number of lineal feet in exits not less than five feet wide and opening outwardly should be $(7 \text{ plus } \frac{\text{capacity}}{60})$.

FIG. 149*b*.
Lines of Sight in Theater.

(5) *Fireproofing*

The many disasters which have occurred in theaters make the protection of the steel and the elimination of the inflammable material extremely important.

(6) *Mechanical Conveniences*

These resemble the corresponding conveniences for office buildings, see Art. 145. Elevators are not usually required and ventilation is much more important and should always be provided to the amount of 2000 cu.ft. per hour for each person when house is filled to its rated capacity.

Building

Either the cage construction or masonry bearing walls may be used.

The building should be fireproof. To the popular mind this conveys an undue sense of safety. A wooden building with ample exits, properly arranged drafts, and an approved fire curtain for the stage is far safer than a fireproof structure without these precautions.† Still the combination of all good points is the best design.

Separating the stage from the auditorium is a fireproof wall of brick, pierced by the *proscenium opening*, Fig. 149*a*.

* See Engineering News, Vol. 70, p. 228.

† See a description of the Iroquois disaster in the Engineering News, Vol. 51, pp. 21 and 34.

The brickwork above, apparently supported by the *proscenium arch*, is in reality borne by a fireproof box girder just above the crown. An average clear span is 40 feet. The depth of the stage is likely to be more, while its width is twice as much. Opposite the proscenium opening, floor is of wood on steel I-beams, this apparent disregard of the fire hazard being for operating reasons. Outside of these limits, only fireproof floors should be allowed.

At the side of the stage on mezzanine floors are located the dressing rooms, extending towards the opening but not projecting from it, Fig. 149*a*. Floors are fireproof, supported

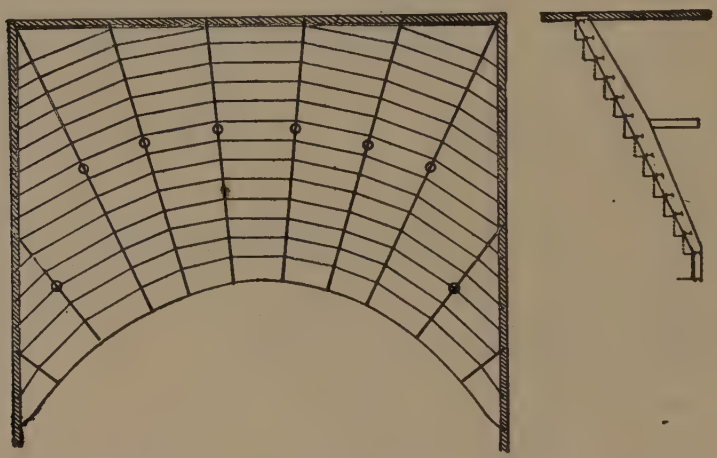

FIG. 149*c*.—Framing for a Theater Floor.

by I-beams, and carry wire and plaster partitions. The purpose of the fly galleries is to manipulate the scenery by means of ropes. It, too, should have a fireproof floor, the edge towards the stage being carried by a latticed or solid plate girder extending from proscenium wall to that in the rear of the stage. It should have pins every 8 inches for attaching stage ropes from which the scenery is suspended. Sometimes a *paint bridge* runs from one gallery to another on the opposite side. The two girders which support it should be well braced together. They may have solid or open webs.

About 7 feet below the bottom of the roof and a little over

twice the depth of the proscenium opening from the stage is the *gridiron*. This is a steel framework over the stage from which the scenery is hung, figured for about 150 pounds per square foot. We may employ either small channels at intervals or a $2'' \times 4''$, $7''$ on centers for the net work, supporting them on a frame work of I beams attached to the roof.

Framing for the Auditorium

The framing for the level portion of the building needs no special attention. However, that which supports the seats is different for two reasons:

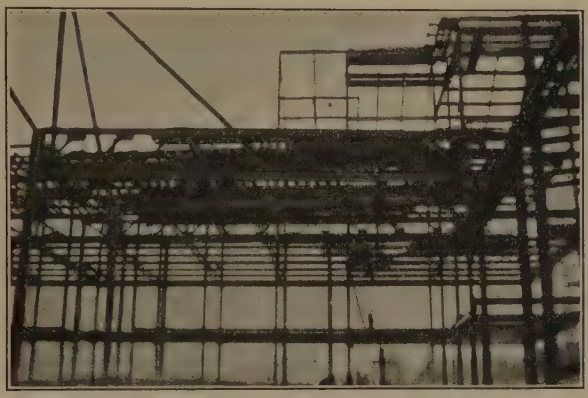

Fig. 149*d*.—Lattice Girder over Theater in School of Applied Design, Carnegie Institute of Technology, Pittsburgh, Pa.

(1) The desirability of freedom from obstruction by columns.
(2) The conical surface to be carried.

The steps on which the seats are placed should be of concrete. Supporting this at intervals of not more than a step apart are the I-beam stringers placed vertically and figured for a live load of 150 lbs. per sq.ft. Fig. 149*c* shows a typical arrangement. Around the edge of the balcony and gallery is placed the curved fascia girder, usually a channel, rolled or built. We know of no reliable formula for computing a beam of this form subject to torsion. It may be figured as straight and a liberal additional margin of safety provided. The radial

floor beams, usually with solid web and variable depth, are expensive in the shop. Depth should be kept small to save head room. Ample provision must be made for live load on cantilever end only. At the same time it is desirable to keep the columns back as far as practicable. Cast-iron posts are favored here. They take little room, are not likely to injure patrons' clothing, and are mildly fire resisting. Occasionally, girders running from wall to wall eliminate columns entirely.

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Canadian Engineer, Vol. 19, p. 10. Toronto, Can.
Engineering News, Vol. 51, p. 21, 34. Iroquois, Chicago, Ill.
Engineering News, Vol. 52, p. 394. Chicago, Ill.
Engineering News, Vol. 54, pp. 642, 631. Fireproofing.
Engineering News, Vol. 59, p. 392. Cleveland, Ohio.
Engineering News, Vol. 63, p. 722. Columbia, San Franscisco.
Engineering Record, Vol. 24, p. 22. McVicker's, Chicago, Ill.
Engineering Record, Vol. 24, p. 23. Metropolitan, St. Paul.
Engineering Record, Vol. 33, p. 61. Tremont Temple, Boston.
Engineering Record, Vol. 40, p. 363. Jersey City, N. J.
Engineering Record, Vol. 42, p. 132. Folly, Brooklyn.
Engineering Record, Vol. 44, p. 342. Boston, Mass.
Engineering Record, Vol. 50, pp. 608, 622, 649. New York City.
Engineering Record, Vol. 51, p. 352. Hippodrome.
Engineering Record, Vol. 57, p. 209. Brooklyn Academy of Music.
Engineering Record, Vol. 58, p. 721. New York City.
Engineering Record, Vol. 59, p. 134. Philadelphia, Pa.
Engineering Record, Vol. 61, p. 741. Slope for seats.
Engineering Record, Vol. 66, p. 468. Palace, New York.
Engineering Record, Vol. 67, p. 52. Harris, Pittsburgh, Pa.
Engineering Record, Vol. 68, p. 433. Church, New York.

Prob. 149. Design a gridiron to be hung from roof trusses extending perpendicularly to curtain and 12 feet apart. 2″ channels with backs up and 4″ c to c., carry the load and are parallel to roof trusses.

Art. 150. Grandstands

These may be regarded as modified theaters, built for spectators of out-of-door events, such as games of base ball and football, track contests, horse races, and so on. We have here the same vital question of capacity. Room will not be as valuable in general and spacing of seats and widths of aisles may be increased a trifle if desired. The same general method of determining heights of seats is available. On account of the somewhat more extended field of view, chairs are not arranged

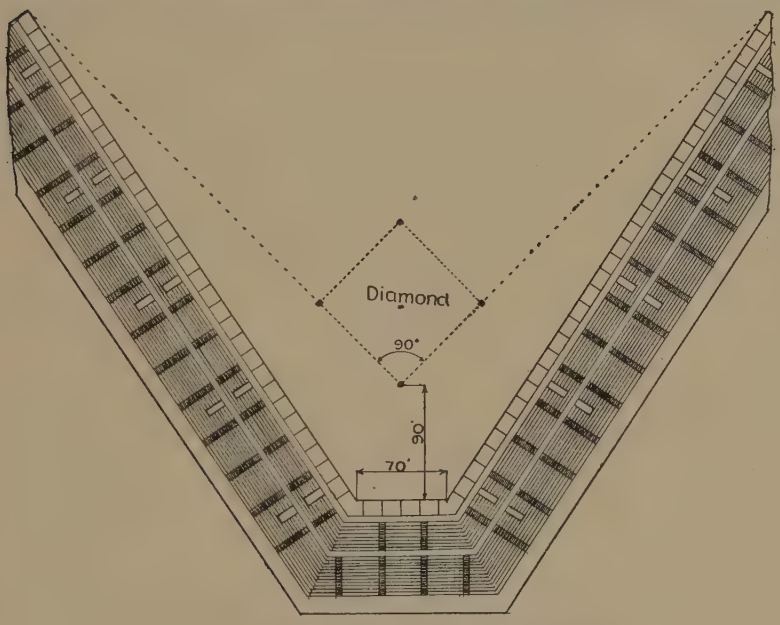

FIG. 150*a*.—Typical Grandstand for a Ball Park.

in circles but along straight lines broken only occasionally. Stand should be located as near field as possible but must not infringe on margin required for play. Hearing is not as important as in the theater and distance of remotest seat is much increased. Ample exits should be provided for fire or panic but need is not so urgent. For the same reason, fire-proofing may be omitted unless there be much wood used in construction or in buildings in the vicinity. It hardly seems

necessary as a safety measure. Mechanical conveniences and mechanism are usually absent.

The best construction is seats of reinforced concrete, the treads being figured as beams carrying loads to the risers extending between steel supports. For the first floor, these might be I-beams resting on posts; or trusses may be employed. The latter method is best for upper tiers, as it decreases the

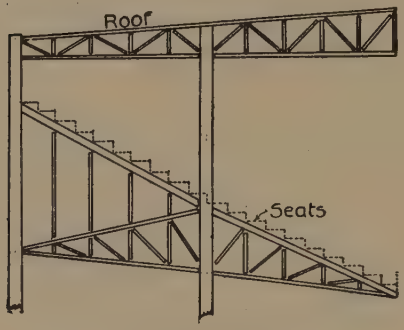

FIG. 150*b*.
Cross-section of the Same.

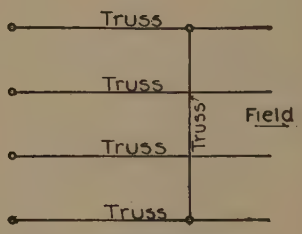

FIG. 150*c*.—Arrangement of Columns and Trusses.

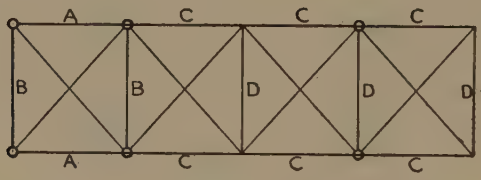

FIG. 150*d*.—Bracing for a Grandstand.

AA, *BB* } Bracing in a vertical plane extending to foundations.
CCCCCC. Chords of projecting trusses.
DDD. Struts between projecting trusses.

number of columns, thus enhancing value of back seats, Fig 150*c*. Columns will often be long and isolated; compactness is desirable yet stiffness and ability to resist transverse moment is essential. Two channels, two channels and an I, or a built I section may be employed. The trusses are usually composed of members with a section consisting of two angles, and somewhat resemble roof trusses, see Fig. 150*b*.

The question of bracing is evidently one of great importance. Wind should be supposed to act on all exposed surfaces of *all* trusses, columns, beams, roofs, and floors, and liberal pressures allowed. A stiff floor or roof (those of concrete for example) properly attached, may be assumed to constitute a bracing in its own plane. There are two methods of providing for the wind.

(1) Knee bracing.

(2) Fig. 150*d*. This is applicable only when there are rooms or passage ways in back. Rods should not be employed for this bracing.

REFERENCES

Engineering News, Vol. 23, p. 57. Monmouth, N. J.
Engineering News, Vol. 60, p. 208. Dallas, Texas.
Engineering Record, Vol. 41, p. 133. Yonkers, N. Y.
Engineering Record, Vol. 51, p. 461. Belmont Park.
Engineering Record, Vol. 64, p. 126. New York Baseball Grounds.
Engineering Record, Vol. 67, p. 274. Brooklyn " "

Prob. 150. In Fig. 150*d*, let the horizontal load be 150 lbs. per lin.ft.; the distance between trusses 15 feet and each panel length 20 feet. The height of truss above foundations is 30 feet. Find stresses and design members.

Art. 151. Assembly Halls

There are two conditions which call for structures of this sort: first, provision for a meeting place for many persons under one roof; and second, to furnish a large enclosed space. Such are convention and exposition halls, armories, and gymnasiums. They are occasionally combined with grandstands.

Usually these buildings have but one floor located at or near the ground. Span may vary from 100 to 200 feet, or even more. There are three types of construction:

(*A*) *The Arch Steel Frame*, Fig. 151*a*. This consists of an arch structure supporting roof and resting on foundations.

Notice that siding is also carried by it. We may recognize three subdivisions: $A0$, the continuous arch; $A2$, the two hinged; $A3$, the three-hinged.

(*B*) *The Knee-braced Steel Frame*, Fig. 151*b*. This type is similar to ordinary mill-building work, see Chapter X.

(*C*) *The Masonry Side-wall Building*, Fig. 151*c*. These are much like the buildings described in Art. 148.

Really the latter two are also arches unless one end is made frictionless. So considered (*A*) constitutes the best design and is the accepted type. Conforming to the usual American practice, the three-hinged arch is most widely employed. To investigate the economical rise, let us consider a span, L, of

Type *A*. Fig. 151*a*. | Type *B*. Fig. 151*b*. | Type *C*. Fig. 151*c*.

Construction for Assembly Halls.

rise, R, as uniformly loaded with w per lin.ft. Equation of the curve considered as a parabolic arch is, Fig. 151*d*:

$$y = 4Rx^2/L^2$$

Letting θ represent angle of inclination to OX, the stress at any point, x, is, Art. 137.

$$wL^2 \sec. \theta/8R$$

The corresponding length is $\delta x \sec. \theta$. Multiplying this by the stress and substituting for $\sec^2 \theta$, $1+\left(\frac{\delta y}{\delta x}\right)^2$, we get

$$\frac{wL^2}{8R}+\frac{8wRx^2}{L^2}\ \delta x.$$

Integrating between the limits, $L/2$ and 0.

$$\frac{wL^3}{16R}+\frac{wRL}{3},$$

as the sum of the products of the stress times length for the

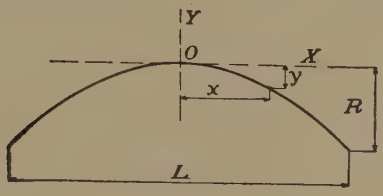

FIG. 151*d*.—A Parabolic Arch.

half span. The value of R which renders this a minimum is obtained by differentiation to be $0.43L$. Although there are several things which have not been taken into account, notably increase of wind pressure with the height, above results agree very well with values used in practice.

Confining ourselves now to the $A3$ type, we find trusses spaced much farther apart than mill buildings as indeed we might infer from the longer spans. 40 feet may be named as an average value. For such bays, light latticed girders, Fig.

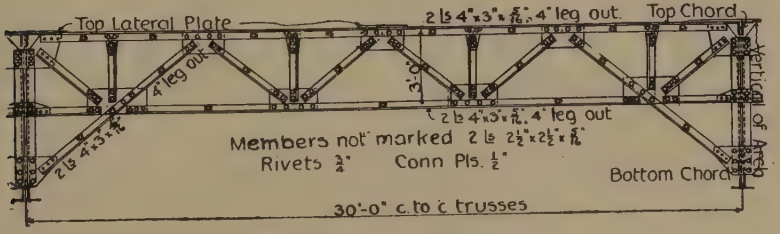

FIG. 151*e*.—A Latticed Purlin.

151*e*, may be employed to advantage for the purlins. This form also acts as sway bracing for the trusses. We may use systems of roofing mentioned in Art. 124. For such long spans, weight will be an important factor in the choice.

A typical truss is shown in Fig. 151*f*. The upper and lower chords may be of two angles, two angles and a web, or two channels. Any may be reinforced by covers. The usual section for web members is two angles. Pin-connected trusses have been employed, but it is neither customary nor advisable.

If foundations of unquestioned integrity are available at a reasonable cost, they should be made to carry the thrust.

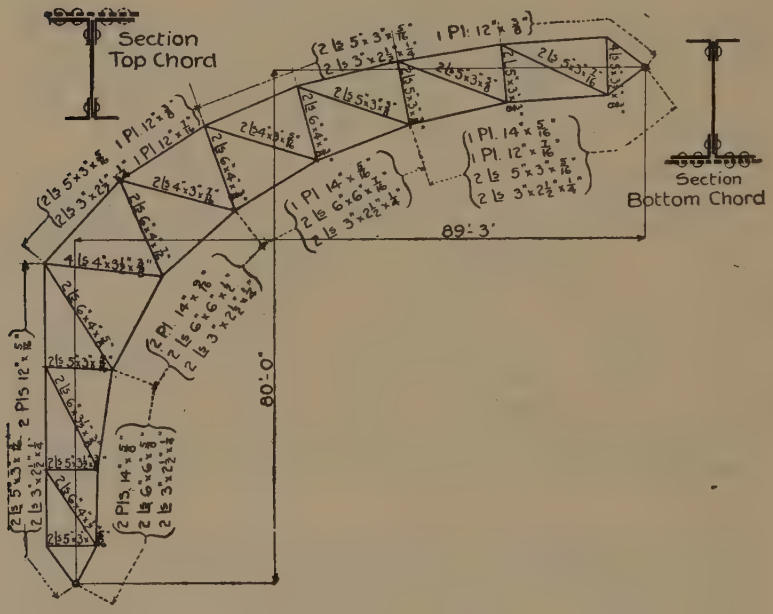

Fig. 151*f*.

A Typical Three-hinged Arch Truss.

Otherwise one hinge will be placed on rollers and the two united by a loop rod just below the floor. Or some other structural shape may be substituted, an I-beam or a channel for example.

Two systems of bracing are necessary; one in the plane of the top chord; the other is the sway bracing already referred to.

REFERENCES

Periodical.	Vol.	Page.	Type.	Span, Feet.	Location.
Engineering News...	58	220	Ao	179	Armory, N. Y.
Engineering Record.	67	210	Ao	124	Grand Central, N. Y.
Engineering News...	55	716	A2	198	Chicago, Ill.
Engineering Record.	49	604	A2	190	Baltimore, Md.
Engineering News...	28	194, 218	A3	368	Columbian Exposition
Engineering Record.	41	548	A3	227	Buffalo, N. Y.
Engineering News...	36	306	A3	215	Coliseum, Chicago
Engineering News...	70	1182	A3	206	University of Illinois
Engineering and..... Contracting	40	159	A3	206	University of Illinois
Engineering News...	53	566	A3	190	Armory, New York
Engineering Record.	58	291	A3	181	Armory, Hartford, Conn.
Engineering News...	42	84	A3	178	Coliseum, St. Louis
Engineering News...	42	287	A3	172	Exposition, St. Louis
Engineering Record.	55	474	A3	167	Armory, Providence, R. I.
Engineering Record.	41	500	A3	164	Armory, Newark, N. J.
Engineering News...	32	176	A3	156	Armory, Chicago, Ill.
Engineering News...	42	162	A3	150	Coliseum, Chicago
Engineering News...	63	520	A3	134	Armory, New York
Engineering News...	27	649	A3	130	Columbian Exposition
Engineering News...	29	434	A3	115	Columbian Exposition
Engineering Record.	58	215	A3	100	Gymnasium, Syracuse
Engineering News...	27	448	B	115	Columbian Exposition
Engineering Record.	60	78	B	97	Pavilion, Denver, Col.
Engineering Record.	36	474	B	52	Pier, New York
Engineering News...	42	287	C	187	Hall, Kansas, City
Engineering Record.	67	301	C	180	Arsenal, Buffalo, N. Y.

Prob. 151. Design and draw up latticed purlin similar to Fig. 151*e* for allowable stresses for soft steel, Art. 131, when load per lin.ft. is 600 lbs., span 40 feet, depths 5 and 10 feet.

Art. 152. Railroad Stations

We may divide this structure into two parts: the office building and the train shed. The former, like high office buildings, may be divided into Type (1), Load-bearing walls; (2), Independent walls; (3), Steel cage construction. The first will be employed in all but the larger cities. In any case, methods given in this and former chapters should suffice or its solution.

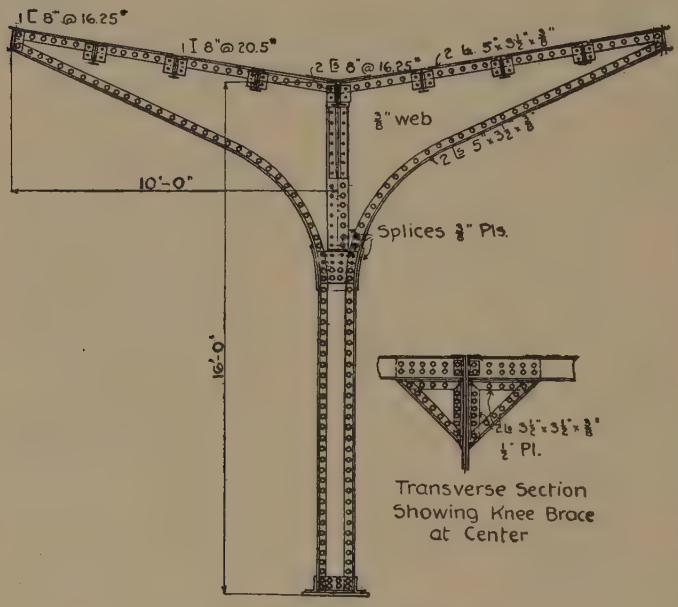

FIG. 152*a*.—Section of Umbrella Shed.

For the trainshed, the general methods of the preceding article are available and we have Types $A0$, $A2$, and $A3$, arches; Type B, knee-braced steel frame; Type C, masonry side wall. These are all costly and difficult to ventilate and maintain, the corrosive gases of the locomotive being particularly hard on steelwork. Such objections have led to Type (4), the umbrella shed.

There is little new in regard to the buildings or the arches. The latter may have somewhat larger spans than for assembly halls. On this account, we are more likely to find pin-connected trusses.

Umbrella Sheds

The usual arrangement is to have each shed supported by a single row of columns. The roof projects only just enough to protect the passengers as they alight. The principal stress on the shed is that of the wind in a direction perpendicular to the track. An I section is best suited for the posts. The projecting beam should be built on the idea of a sidewalk bracket. The design of Fig. 152*a* represents a typical arrangement. Perpendicular to the paper, provision for stiffness is made by

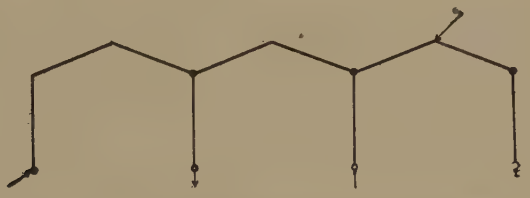

Fig. 152*b*.—Proposed Continuous Umbrella Roofs.

the use of four angles latticed together, or two knee-braced channels as shown. It may be figured as a continuous portal, Art. 99. For computations in this direction, we may consider the wind to be inclined and causing a pressure of 5 pounds per sq.ft. of actual roof area plus 40 lbs. per sq.ft. on exposed surface of each entire post. The design is often modified somewhat by the desire for architectural embellishment.

By uniting these roofs as is sometimes done, we obtain a series of continuous arches. This is the usual construction. The uncertainties of stress and difficulty of computation may be removed by inserting joints as seen in Fig. 152*b*. In this case, left end reaction carries entire thrust.

REFERENCES

Umbrella Roofs in General, Engineering Record, Vol. 67, p. 6.

Period.	Vol.	Page.	Trainshed.		Station.		Location.
			Type.	Span.	Rise.	Type.	
Engineering News...	41	25, 348					General
Engineering News...	50	114	A0	207	46	3	Chicago, Ill.
Engineering Record.	43	475	A1	46			Des Moines, Iowa
Engineering News...	42	212	A2	253	92	3	Jersey City, N. J.
Engineering Record.	68	666		91	36		Rochester, N. Y.
Engineering News...	26	276	A3	253	90		Jersey City, N. J.
Engineering Record.	46	172	A3	255	92		Pittsburgh, Pa.
Engineering News...	29	50, 98	A3	259	88		Philadelphia, Pa.
Engineering News...	29	507	A3	301	108		Philadelphia, Pa.
Engineering News...	32	381	B	87		3	Brooklyn Bridge, N. Y.
Engineering News...	34	2	B	89		3	Brooklyn Bridge, N. Y.
Engineering Record.	44	242	B	126			Camden, N. J.
Engineering News...	27	394, 436	B	141			St. Louis, Mo.
Engineering Record.	48	302	B	212		3	Chicago, Ill.
Engineering Record.	39	134	B	229			Boston, Mass.
Engineering Record.	61	774	D			3	Chicago, Ill.
Engineering News...	56	297	D			3	Hoboken, N. J.
Engineering News...	41	278, 342				1	Omaha, Neb.
Engineering Record.	61	675				2	Winnipeg, Can.

Prob. 152. Design an umbrella shed 300 feet long for tracks 26 feet center to center. Height of car, 15 feet; width 10 feet. Use soft steel and specifications for mill buildings, Art. 131.

Art. 153. Mine Structures

These may be divided into two types:

(*A*) Those for supporting the hoisting apparatus, the *head frames.*

(*B*) Those containing the machines for treating the materials and delivering them for transportation. These processes vary with the product handled, and even for the same mineral, with its nature. For example, coal may be broken, washed, screened and stored. In any event, building must be planned largely to accommodate these processes.

Head Frames

The angle i will vary from 20 to 75°, the mean position, about 40°, being best for the usual case. The principal forces

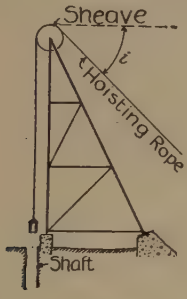

Fig. 153*a*.

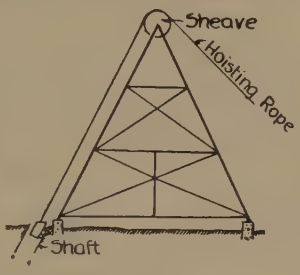

Fig. 153*b*.

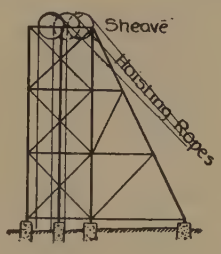

Fig. 153*c*.

Typical Head Frames.

acting on the frame are the two pulls on the rope, obviously equal. The best arrangement for withstanding this is the A frame seen in Fig. 153*a*.

It is better here as elsewhere if resultant can be arranged to always fall within the base, thus eliminating tension on foundations. The latter must not be placed so near the openings that there is danger of a cave in. For an inclined shaft, the vertical A frame may be modified as seen in Fig. 153*b*. Transversely, frames may have parallel posts or be battered. The latter is better theoretically but adds to the cost of detailing and shop work. Where hoisting from several compartments not in line, method of support seen in Fig. 153*c* is common. It

would seem better to remove one set of diagonals, thus rendering structure determinate.

The forces which act upon a head frame are:

(*a*) Its own dead weight, including bearings, pulleys, and so forth.

(*b*) Wind load, which may be taken at 30 lbs. per sq.ft. of exposed surface.

(*c*) The pulls on the rope already mentioned. The latter is taken as equal to the weight of rope plus twice that of the heaviest loaded skip, the coefficient being introduced to provide for instantaneous starting. This is termed the " working load." Another case which must be considered is that where the load sticks and the rope breaks, the " breaking load." We now

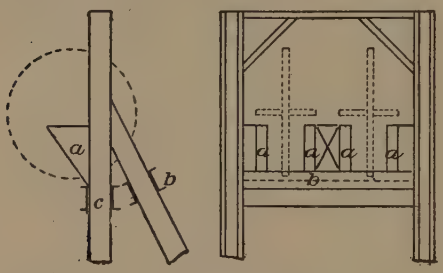

FIG. 153*d*.
Sheave Supports.

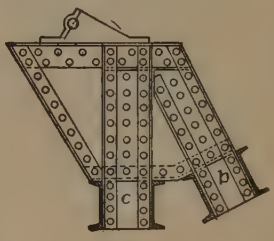

FIG. 153*e*.
Details for Fig. 153*d*.

have the following combinations, using allowable stresses for mill buildings, Art. 131:

(*1*) Dead plus twice the working load.

(*2*) Four-fifths of (dead plus twice working load plus wind).

(*3*) Dead plus two-thirds of breaking load.

In the above, the coefficients are designed with a view to varying actual stress according to the probability and destructive effect of each load. If there be several hoists, live loads should be for all acting simultaneously.

At the top, supports for the sheaves are provided by cantilevers resting on the sheave girders. The latter may be made of two channels. Figs. 153*d* and 153*e* show a typical arrangement. A more central support and a correspondingly larger sheave might be desirable. The usual section for the posts

is two channels with webs parallel to sheaves. Bracing should possess more or less rigidity. Two channels latticed is common section for struts and two angles latticed or battened for diagonals. The latter are figured, however, as tension members.

Type (*B*), Building

There is a tremendous variety of these structures even in the same line of business. The length of time required to exhaust the mine should be carefully considered in the design. Also investigate the effect of corrosive gases which may be developed.

These structures are essentially mill buildings. Corrugated steel, Art. 124, is common for roofing and siding, as indeed might be inferred from their temporary nature, and from the serious consequences of fire, especially in coal mining. There is nothing new in the design of the trusses, which are usually of moderate span. Independent support for head frame and building are advisable, especially in case the latter contains shaking screens. Column sections are rather light—a single angle, two angles, the built I, two channels latticed, and Z bar. The first two seem rather deficient in transverse strength. This is quite important, since structure is much exposed and often contains machinery. Particularly severe upon the building are the above-mentioned screens. Where either of these factors are present, rod bracing should not be allowed, one, two, or four angles latticed together being the proper design. Floors are of timber on steel I-beams. Loads and specifications are the same as for mill buildings except for the provision for machinery, which varies much for each individual case. The design of bins, which are often a part of the building, will be considered in Vol. III.

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Canadian Engineer, Vol. 24, p. 323.

Prob. 153. A vertical shaft with two compartments has a hoist with the center of the sheaves 60 feet high. Weight of one loaded hoist, 12,000#; rope for same, 3000 lbs., breaking load of rope 100,000 lbs. Rope to the hoisting engine makes an angle with the horizontal of 45°. Sheaves will be 8 feet diameter, located 7 feet apart and in line. Foundations will not be allowed within 12 feet of either hoisting rope. Wind pressure to be computed for 30 lbs. per sq.ft. on the basis of enclosed structure. Design head frame and compute weight, using specifications of Art. 131, modified as noted above.

Art. 154. Miscellaneous Buildings

Pier Sheds

These are very much like the typical mill building. Spans are likely to be longer, also on account of doors, possible uplift due to wind needs close attention. A little more heed is paid to architectural appearance and loading and unloading devices rather than cranes should be considered.

Grain Elevators

An elevator is a building for the purpose of raising, cleaning, weighing, and storing grain. It may be of timber, steel, concrete, tile, brick or reinforced concrete. We shall consider only the second. Fig. 154 represents a typical installation.

Roofing, siding and roof trusses are much as in mill buildings. On account of the height, beams, columns, and wind bracing resemble those in high office buildings.

Bins are usually circular in shape. Space between them may be used for storage, either by arranging to fill vacant space with grain, or by inserting smaller cylindrical bins. In computing, the possibility of one being entirely empty when

adjoining bins are full, or the reverse, must be considered. In general, construction resembles standpipes. The theory of their design, which is quite complicated, will be considered in Vol. III. Specification in most respects will resemble those for mill buildings.

Power Stations

Power stations in general are masonry buildings with structural steel roofs. Cranes are almost a necessity; it is then better if there be columns in the wall. This type of building was considered in Art. 148.

Coal Storage

Coal in large quantities is handled by machinery. Elevators are used for a vertical movement. These are buckets fastened to an endless chain on either side, which pass over sprocket wheels at each end. For horizontal travel, conveyors are employed. Their construc tion may be similar to elevators, or simple vertical plates may be used instead of buckets, acting to push along the material. Or it may be transported by plain belts, dished to prevent spilling. Bituminous coal is liable to spontaneous combustion. To prevent or minimize loss, we may insert fireproof partitions, provide for rapid removal in emergencies, and limit depth to about 25 feet. The slope down which coal will slide is 35 to 40°. Weight in pounds per cubic foot, loose, are as follows: anthracite, 60; bituminous, 50; coke, 30. With these few elementary facts, we must dismiss a subject which could fill a volume. The design of bins, together with the theory of the pressure of granular materials will be covered in Vol. III.

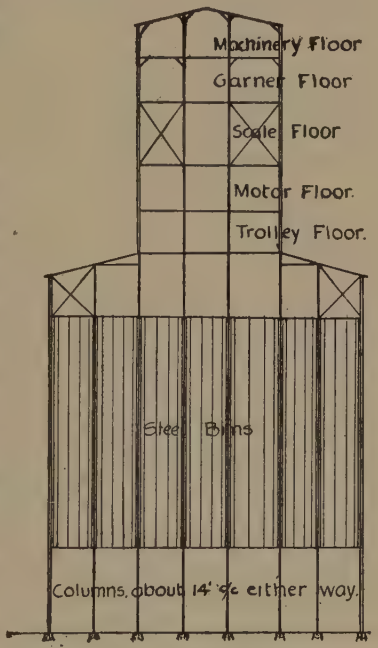

Fig. 154.—A Typical Grain Elevator.

Large storage plants for coal are usually out-of-doors, rest on ground floors, and the installation is mainly a question of mechanical engineering. If, due to an inclement climate, a cover is to be provided, it will be much like those for assembly halls, Art. 151. Smaller storage plants are often provided by consumers to protect against fluctuations in supply. These are usually bins, often carried and braced by structural steel. They will also be considered in Vol. III.

Miscellaneous

Among the other types to which steel is applicable, we find space for but a brief mention of two of our references, the barn and the balloon house, both rather unusual, but probably to be much more common in the future.

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Pier Sheds

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Grain Elevators

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Engineering Record, Vol. 44, p. 620.

Power Stations

Engineering Record, Vol. 40, p. 690; Vol. 44, p. 35; Vol. 45, pp. 244, 249. Vol. 50, p. 456; Vol. 52, p. 375.

Buildings for the Storage of Coal

Engineering News, Vol. 38, p. 422; Vol. 40, p. 99; Vol. 52, p. 68; Vol; 58, p. 211.
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Miscellaneous Buildings

Engineering News, Vol. 60, p. 217. Conservatory.
Engineering Record, Vol. 45, p. 180. Earthquake proof houses.
Engineering Record, Vol. 52, p. 655. Ash plant.
Engineering Record, Vol. 58, p. 194. Barn.
Engineering Record, Vol. 60, p. 637. Balloon house.

CHAPTER XIII

STAND-PIPES AND ELEVATED TANKS

Art. 155. General

THESE structures are usually adjuncts to systems of water distribution. They equalize pressure and afford a reserve capacity in case of extraordinary demand as in a fire or a break-down of some part, for example, the pump. In a small town the latter may run only during the day, relying on the storage thus provided for the remainder of the time. For a further discussion of the hydraulic questions involved, see any of the numerous works on this subject.

A *stand-pipe* is a round vertical pipe used for the storage of water, oil, or some other liquid. It is open at the top, even if roofed, since latter is not air tight. Then, the pressure at any point is that due to the hydraulic head. For some locations, notably in fairly level towns, the lower part of the stand-pipe will not give sufficient pressure. In this case it is often better and cheaper to replace its lower part by columns, usually four or six in number, braced together and supporting what is virtually a small stand-pipe. This arrange- is termed an *elevated tank.*

The cost of stand-pipe work will be about base price of steel plus transportation plus 2 cents per pound. For the columns the latter figure may be replaced by $3\frac{1}{4}$ cents. These prices will vary somewhat with the height.

ERECTION

For a stand-pipe, the flat bottom is first riveted up and then lowered on the grouted surface of the masonry. On the sides the plates are likewise shipped separately and field riveted, each course being first bolted up. Plates are hoisted by means of a short gin-pole, bolted to top row of tank. After riveting, joints must be *calked* on the inside. At the shop, they are first

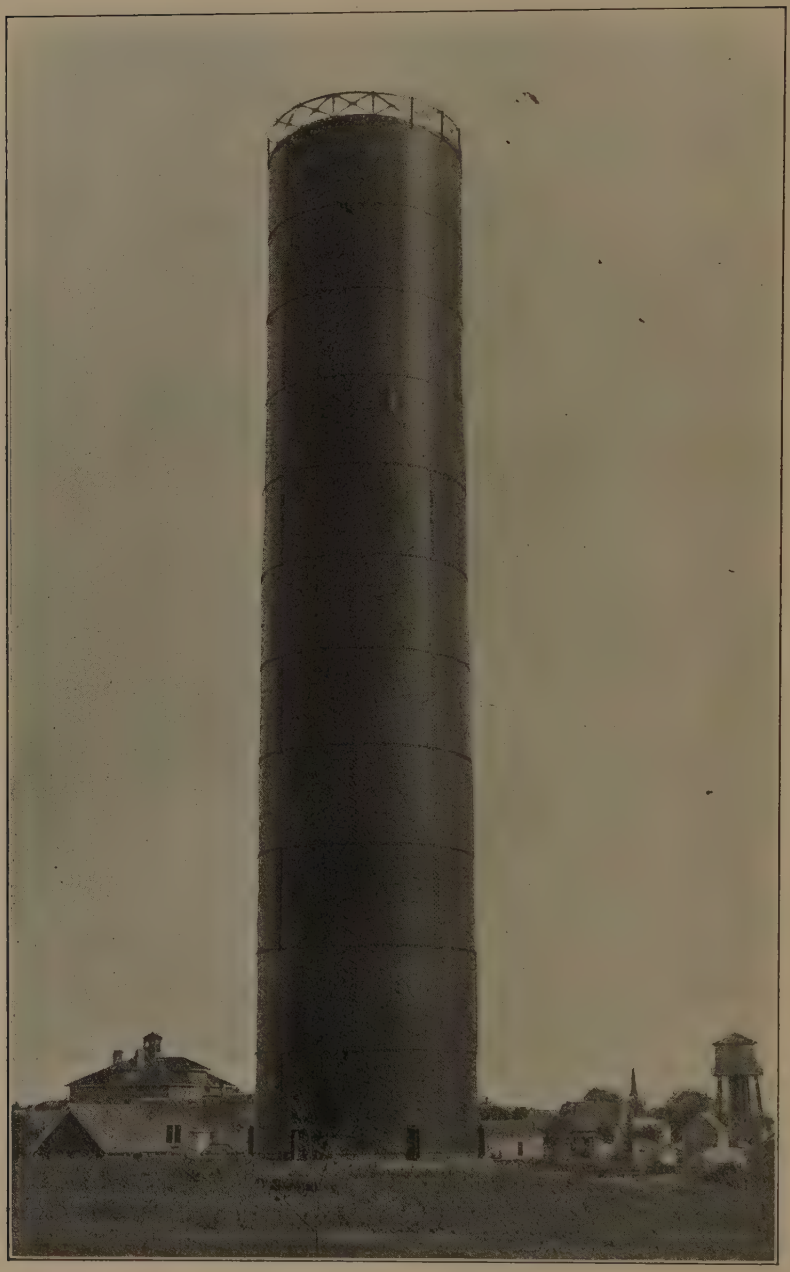

FIG. 155*a*.—Typical Stand-pipe, Creighton, Neb.
16 feet diameter, 80 feet high.
Des Moines Bridge & Iron Works, Pittsburgh, Pa.

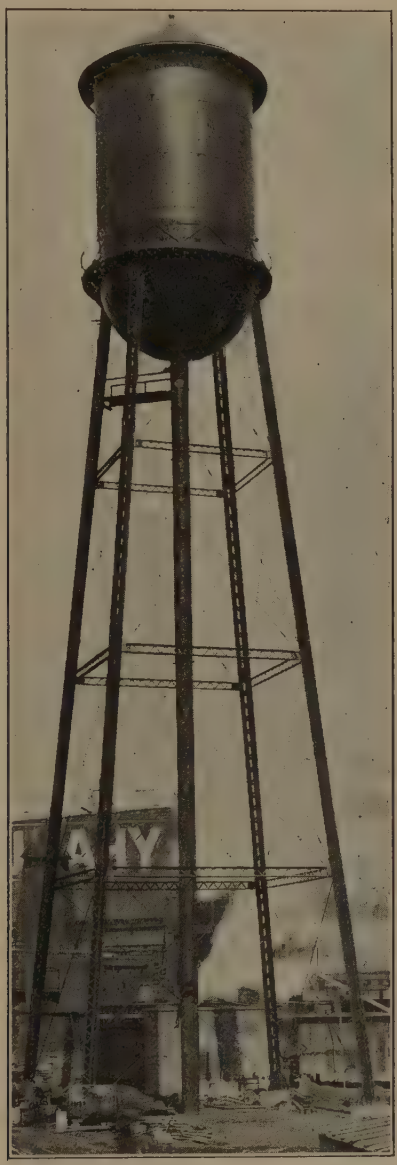

FIG. 155*b*.—Standard Water Tower for Industrial Fire Protection, Kansas City, Kansas. 100,000 Gallons Capacity. Height to Top, 144 feet.

beveled as shown in Fig. 155*c*. The calking tool, a rounded chisel, compresses the steel into the joint, thus rendering it water tight.

The same general method is followed in elevated tanks, the material being erected by gin-poles fastened to that already finished. The towers are erected one story at a time.

FIG. 155*c*. Calking a Joint.

FAILURES

These structures, more particularly the standpipes, seem to be especially liable to failure. The percentage here is probably higher than for anything else built of steel to carry loads which are largely quiescent. These accidents are not always easy to explain and sometimes occur in a standpipe which would seem to be well designed. The principal causes are:

(*1*) The use of an *inferior grade of brittle steel.*

(*2*) *Wind*, causing a local buckling, or an overturning as a whole.

(*3*) *Ice.* A large mass may fall, due to the melting underneath, confined water may freeze, or pumping into a tank closed by ice may cause an abnormal pressure head.

In addition to the above causes, the long slender lap-riveted shell carrying some compression under dead and wind loads, seems to be lacking in strength as a column. Possibly this may explain some of the apparent weaknesses of this type of structure. Circular stiffening angles around the pipe, perhaps with the addition of vertical stiffeners, ought to help a great deal. Their use, however, is not common.

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Engineering Record, Vol. 27, p. 216.

ERECTION

Engineering News, Vol. 35, p. 18.
Engineering Record, Vol. 37, p. 362.

REFERENCES FOR FAILURES

Engineering News, Vol. 20. p. 271; Vol. 29, p. 242; Vol. 31, pp. 276, 299, 317, 340, 358, 393, 422, 462; Vol. 33, pp. 267, 285.
Engineering Record, Vol. 27, p. 216.
See also the references for the two following articles.

Art. 156. Stand-Pipes

Thickness of pipe is determined by the well-known formula:

$$t = \frac{pd}{2S_t},$$

where t = thickness of pipe in inches;
p = pressure of water in lbs. per sq.in.;
d = diameter in inches;
S_t = allowable stress in lbs. per sq.in.

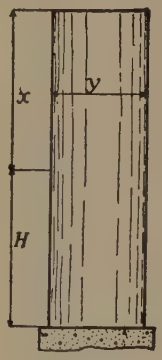


FIG. 156a.
Stand-pipe.

A more convenient form may be obtained by substituting for p its equal, 0.434h, where h equals head in feet, changing d into diameter in feet, and substituting for S_t, 8000, which is about the common value when efficiency of the joint is considered. We then have:

$$t = \frac{hd}{3200}, \; t \text{ in inches, } h \text{ and } d \text{ in feet.}$$

The problem is usually to install a given capacity above a given height at a minimum cost.

Let it be required to determine economic dimensions, x and y, above the height H, Fig. 156a, to hold a certain quantity of water, Q. Since the cost, E, varies with the thickness of

metal and diameter, the former in turn varying with the diameter, we may say,

$$E = C_1 y^2.$$

Similarly,

$$E = C_2 (H + x)^2,$$

or

$$E = C y^2 (H + x)^2,$$

but

$$Q = \pi x y^2 / 4 \quad \text{or} \quad x = 4Q/\pi y^2.$$

Substituting,

$$E = C\left(Hy + \frac{4Q}{\pi y}\right)^2.$$

Differentiating for a minimum, we obtain

$$H = \frac{4Q}{\pi y^2} = x.$$

However, this proof fails to take into account cost of masonry and bottom plate and effect of wind pressure.

For the case where $H = 0$, that is, where all storage is available, thickness of plate will be about the minimum. We may then say:

$$E_1 = \pi C_3 x y,$$

where C_3 equals cost per sq. unit of surface of pipe. The expense of the foundations and bed plate now becomes quite important. Let us call the combined cost per sq.ft. as C_4. Then,

$$E_2 = \frac{\pi}{4} C_4 y^2.$$

As before

$$Q = \frac{\pi x y^2}{4} \text{ or } x = 4Q/\pi y^2$$

Total

$$E = \pi C_3 xy + \frac{\pi}{4} C_4 y^2 = \frac{4C_3 Q}{y} + \frac{\pi}{4} C_4 y^2,$$

$$\delta E / \delta y = -\frac{4C_3 Q}{y^2} + \frac{\pi}{2} C_4 y = 0,$$

whence

$$y = 1.36 \sqrt[3]{\frac{C_3 Q}{C_4}}.$$

For the usual case where there are good foundations, the results will usually make x a little less than y. The first case might be solved similarly, by taking foundations into account, but we believe that the relation deduced is best because cost of pumping and provision for stability under wind pressure tend to neutralize the other factor.

The above proof might be applied to elevated tanks if C_4 be allowed to represent increased cost of bracing and pedestals when area of tank is increased one square unit.

In general a stand-pipe is made of curved plates of a height sufficient to lay about 5 feet net, of a length along circumference of about 10 feet, and of computed thickness. However, the latter ought not to be less than $\frac{1}{4}''$. More than $1\text{-}1/8''$ should be avoided.

Fig. 156*b*. Scarfing.

Vertically the stress is small and the horizontal joints are single lap riveted. One method is to make alternate rings of different diameters, the two sets varying by twice the thickness of the metal. The other way is to cone the plates slightly so that upper row goes over the one below. The latter quite evidently adds to the shop cost, but enables the calking to be done from above. The vertical joints suffer a considerable stress and are likely to vary somewhat with the thickness of the plate. However, there should be nothing less than double-riveted lap joints. The diameter of the rivet is likely to vary similarly with $\frac{5}{8}''$ for a minimum. For lap joints and butt joints with a single strap the greatest theoretic efficiency is

obtained when diameter equals 2.5 times thickness of plate. Just how far this should be allowed to affect the practical advantages of similarity of joints and equal-sized rivets is a question for the designer to determine. A solution is attempted in Fig. 156*g*. A compromise is usually best. Two practical points claim our attention. The spacing of rivets near an edge to be calked, in order to secure tightness, should not exceed 10 times thickness of plate. Also with lap-riveted joints, the union of three plates has an additional thickness. To prevent the awkward appearance in the finished pipe, the overlapping corners are drawn out by the blacksmith, Fig. 156*b*. This process is termed " scarfing."

The bottom plate, the " bed plate," is made up of segmental pieces. While theoretically it carries no stress, it should be

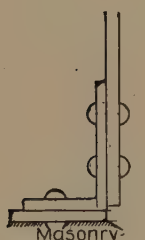


FIG. 156*c*.
Bottom Angle.

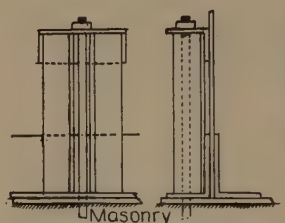


FIG. 156*d*.
Uplift Bolts.

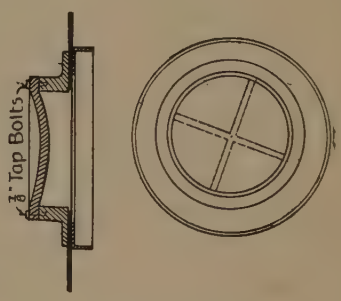


FIG. 156*e*.
Manhole.

about the thickness of the lower ring. Joining it to the upright pipe, is an angle, Fig. 156*c*. The thickness of the latter should be about equal to that of the plates which it joins. The length of the legs is determined by the character of the riveting required; that is, whether single or double riveting. It should be firmly bolted to the foundations.

If there is a possibility of an uplift, bolts must be figured. For high stresses, a detail should be arranged as in fixed-ended columns, see Fig. 156*d*. At the bottom, a manhole with a clear opening of about $12'' \times 16''$ should be provided, Fig. 156*e*.

Extending up the outside of the stand-pipe and down the inside is a ladder. The first few feet of the former should be omitted to prevent unauthorized use. The latter should be employed only in localities where ice is not likely to form in large masses. It may be made of two bars, $2'' \times \frac{3}{8}''$, $15''$ apart in the clear united by $\frac{3}{4}''$ rungs, spaced $12''$ on centers. These are shipped in sections and field riveted to the shell by angles, about every 20 feet, Fig. 159.

FIG. 156*f*.
Details of Cornice.

An angle should always be placed at the top to stiffen against a wind pressure from the outside. To determine its size, we may use the empirical rule that the angle be figured as a simple beam for a horizontal load of 50 lbs. per lin.ft. for the diameter of the pipe. A Z bar, fastened as shown in Fig. 156*f*, is a great convenience for painting.

There is usually no roof. One is desirable, however, to

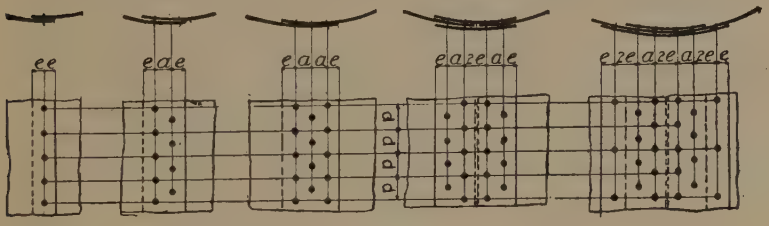


FIG. 156*g*.—Joints for Stand-pipe.

(*A*) Field Rivet Values. (*B*) Shear, 9000 lbs. (*C*) (*D*) (*E*) Bearing, 18,000 lbs. per sq.in.

prevent pollution from birds and to lessen danger from ice. The surface may be conical or curved inward. In either case, it is of plate about $\frac{1}{8}''$ thick, supported by light angle or channel rafters bent at the pipe and fastened to it for a foot or two. Rafters are sometimes omitted for diameters less than 20 feet. It should be provided with a trap door about two by three feet in size. A cornice as shown in Fig. 156*f* may also be added for architectural effect.

The following table represents economical practice for

vertical joints. It may, however, be desirable to modify slightly to keep rivets alike. Type *A* may be used for horizontal joints by those engineers who do not care for full development of the capacity of the plate. Rivet holes are considered to be $\frac{1}{8}''$ greater than nominal diameter. Minimum spacing, 3 diameters of rivet; maximum for a calked edge, 10 diameters.

Rivet.	Shear.		Bearing on Plate.												
	Single.	Double.	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1″
$\frac{5}{8}''$	2760	5520	2810	3520	4220	4920	5620	6330	7030						
$\frac{3}{4}''$	3980	7960	3370	4220	5060	5910	6750	7600	8440	9280	10120				
$\frac{7}{8}''$	5410	10820	3940	4920	5910	6890	7870	8860	9840	10830	11810	12800	13780		
1″	7070	14140	4500	5620	6750	7870	9000	10120	11250	12370	13500	14620	15750	16880	18000

Values below left hand heavy line are less than single shear.
Values above right hand heavy line are more than double shear.
Values between are more than single shear but less than double.

Thickness Plate.	Size Rivet.	Type.	*p.*	*a.*	*e.*	Efficiency.
$\frac{1}{4}''$	$\frac{5}{8}''$	*B*	$2\frac{1}{2}''$	$1\frac{3}{4}''$	$1\frac{1}{8}''$	0.700
$\frac{5}{16}''$	$\frac{3}{4}''$	*B*	$3''$	$2''$	$1\frac{1}{4}''$	0.708
$\frac{3}{8}''$	$\frac{7}{8}''$	*B*	$3\frac{3}{8}''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.704
$\frac{7}{16}''$	$\frac{7}{8}''$	*C*	$4''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.750
$\frac{1}{2}''$	$\frac{7}{8}''$	*C*	$3\frac{3}{4}''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.721
$\frac{9}{16}''$	$\frac{7}{8}''$	*D*	$3\frac{5}{8}''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.724
$\frac{5}{8}''$	$\frac{7}{8}''$	*D*	$3\frac{5}{8}''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.724
$\frac{11}{16}''$	$\frac{7}{8}''$	*D*	$3\frac{5}{8}''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.724
$\frac{3}{4}''$	$\frac{7}{8}''$	*D*	$3\frac{3}{8}''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.704
$\frac{13}{16}''$	$\frac{7}{8}''$	*E*	$3\frac{1}{4}''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.765
$\frac{7}{8}''$	$\frac{7}{8}''$	*E*	$3''$	$2\frac{1}{2}''$	$1\frac{1}{2}''$	0.750
$\frac{15}{16}''$	1″	*E*	$3\frac{5}{8}''$	$3''$	$1\frac{3}{4}''$	0.776
1″	1″	*E*	$3\frac{1}{2}''$	$3''$	$1\frac{3}{4}''$	0.757

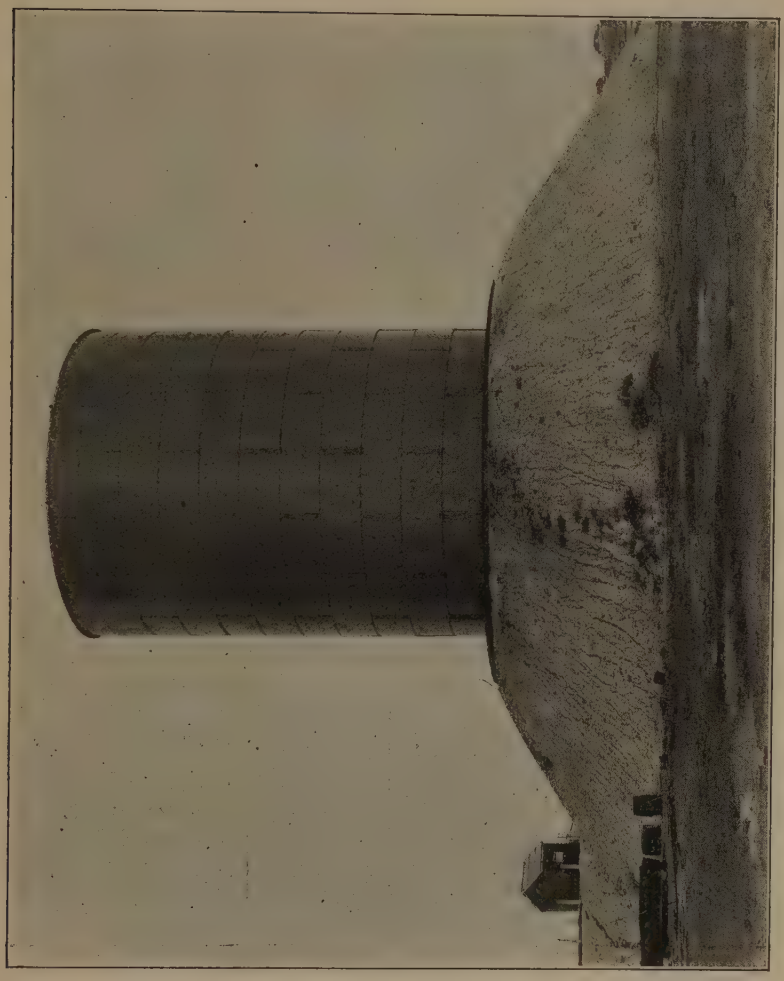


FIG. 156*i*.—Standpipe at San Diego, Cal., 40 feet dia., 52′–6″ high.

REFERENCES

Listed in order of height

Periodical.	Vol.	Page.	In Feet. Height	In Feet. Diam.	Location.
Engineering News...	56	499	20	150	Rochester, N. Y.
Engineering News...	61	350	40	65*	Parkersburg, W. Va.
Engineering Record.	17	289	48	100*	Cincinnati, Ohio
Engineering Con.....	34	482	51	66	Syracuse, N. Y.
Engineering Record.	32	473	65	20	Rhinelander, Wis.
Engineering News...	34	92	70	25	St. Charles, Mo.
Engineering News...	62	472	80	20*	White Plains, N. Y.
Engineering News...	52	507	80	40*	Sanford, Me.
Engineering News...	43	282	95	30*	Elgin, Ill.
Engineering Record.	41	59	100	12*	Collingwood, N. J.
Engineering News...	47	390	100	25*	Lincoln, Neb.
Engineering News...	27	346	100	30	Des Moines, Iowa
Engineering Record.	25	156			
Engineering Record.	37	362	105	38	Flushing, L. I.
Engineering News...	31	286 et seq.	120	25*	Peoria, Ill.
Engineering News...	30	205 237	120	40*	East Providence, R. I.
Engineering Record.	67	12	125	25	Catonsville, Md.
Engineering News...	59	681	130	15*	Waterloo, N. Y.
Engineering Record.	32	298	130	20	Chevy Chase, Md.
Engineering Record.	16	643 et seq.	130	30	Seneca Falls, N. Y.
Engineering News...	29	294	135	18.5*	Maryville, Mo.
Engineering News...	67	209	140	20*	Sheboygan, Wis.
Engineering Record.	23	309	140	22*	Defiance, Ohio
Engineering Record.	39	351	145	15*	Murphysboro, Ill.
Engineering News...	69	825	175	16*	Cairo, Ill.

* Failed.

See also references for failures in the preceding article.

Prob. 156. What sized angle should be used at the top of stand-pipe of 20 feet diameter?

Art. 157. Elevated Tanks

We find little difference between the tank of the elevated structure and the stand-pipe. Perhaps we oftener see the top roofed in. Just why this is done is not apparent. The principal new point, however, is the *curved bottom*. Of course this could be flat and carried by I-beams, but this is so much more expensive that it is unusual. The hemispherical bottom with its entire stress carried by direct tension is really much

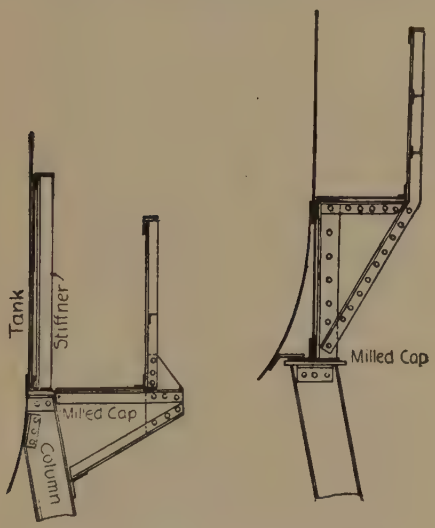


FIG. 157*a*.—Details at Top of Columns.

the best type. However, conical, elliptical, and segmental bottoms are sometimes employed. As the student doubtless recalls from his Mechanics the thickness demanded for a hemispherical end is half of that in the adjacent shell. Many engineers, however, insist that it should have the same thickness. Under no circumstances may less than $\frac{5}{16}''$ be allowed.

The girder which connects the posts at their junction with the tank is necessarily circular. It is figured to carry its share of the load as a straight beam, the twisting being assumed to

be borne by the shell. Proper details are shown in Fig. 157*a*. The balcony should be designed to take the pull from the bottom in case it is curved but not hemispherical. The girder must have stiffener angles over columns, and like any other plate girder, at such frequent intervals that the web will not cripple, Art. 86.

This girder is often omitted, the only provision for the push of the columns against the tank being a local stiffening of the plate. Other designs take care of the horizontal thrust of the columns and of the bottom in case it is not hemispherical.

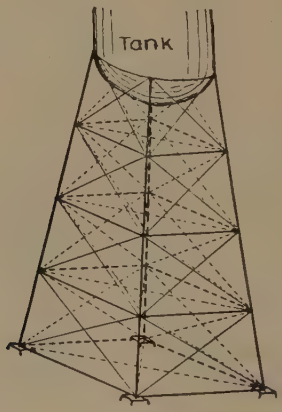


FIG. 157*b*.—Bracing for Tower.

We do not see how the thin, curved, lap-riveted shell may properly be considered as a plate girder to transmit tension of bottom, that is, the weight of tank and water, to the columns. It would seem as though liberty enough had been taken in the assumption that the shell bears the twisting moment.

TOWER

On account of the heavy load, more than four supports are desirable for a large tank. These may be provided by kneeing out from columns on each side to girder, thus tripling reactions. Also the number of the posts may be increased to six or even eight. However this increases cost of material, shopwork, and computations.

These columns are braced together in a manner similar to those for viaducts, Art. 96. They should be inclined, sometimes taking graceful curves for the sake of appearance. The

FIG. 157c.

Elevated Tank at Bristol, Pa. Height to Top 174′. Capacity 150,000 Gallons.

amount of the batter should be enough to prevent a large tension, 1 in 12 being a fair value.

For the posts, two latticed channels are the accepted design, but Z bars may be used. The former is also common

for the struts. The laterals are generally rods with clevis nuts. Column caps, splices, and bases resemble those in typical viaduct work and will not be considered further. One differ-

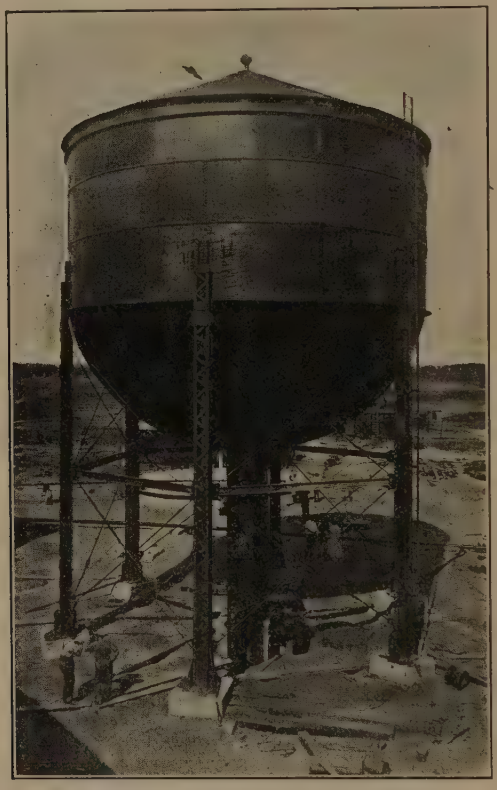


FIG. 157*d*.
Typical Elevated Tank for Railroad Use.

C. M. & S. P. Ry. at Perry, Iowa. 150,000 Gallons Capacity.
Des Moines Bridge and Iron Works, Pittsburgh, Pa.

ence which may be noted in Fig. 157*b*, is the presence of additional sets of bracing in horizontal planes. For towers containing more than four posts, they are quite essential.

REFERENCES

Listed in order of height of pipe.
Dimensions are in feet.

Periodical.	Volume.	Page.	Height Pipe.	Dia. Pipe.	Bottom above Pound.	Location.
Engineering News...	51	475	20	25	70*	Abilene, Texas
Engineering Record.	47	657	33.5	26	200	Camden, N. J.
Engineering Record.	16	641 643 748	35	40	45*	Franklin, Mass.
Engineering News...	65	62	35	30	157	St. Louis, Mo.
Engineering Record.	46	441	50	26	104	Grand Rapids, Wis.
Engineering News...	31	206	40	20	80	Laredo, Texas
Engineering and Contracting	37	202	40	22.5	86	Hamilton, Ill.
Engineering Record.	35	273	41	22	100	Paris, Ill.
Engineering News...	48	544	44	28	101	Chicago, Ill.
Engineering Record.	48	630	45	22	111	Lackawanna Steel Works
Engineering News...	41	258	45	30	100	Jacksonville, Fla.
Engineering News...	34	155				
Engineering News...	46	392	50	35	100*	Fairhaven, Mass.
Engineering News...	46	433				
Engineering News...	47	11				
Engineering Record.	42	6	50	20	102	Murphysboro, Ill.
Engineering News...	39	371	52	24	100	Iowa State Agricultural College
Engineering News...	10	578	60	20	60	Princeton, N. J.
Engineering News...	58	698	70	30	170	Duluth, Minn.
Engineering News...	52	413	70	50	135	East Providence, R. I.
Engineering Record.	50	580				
Engineering Record.	59	732	79	30	102	Washington
Engineering Record.	57	265	90	50	130	Louisville, Ky.

* Failed.

See also references for failures.

Prob. 157. Design circular girder for a six-post tank 20 feet in diameter and 40 feet high from the bottom of the hemispherical base. Use allowable stresses of next article.

Art. 158. Specifications

(*a*) *General*

(*1*) ROOF — If directed by the engineer, a roof shall be furnished consisting of $\frac{3}{16}''$ plates, fastened by $\frac{1}{4}''$ rivets. Rafters shall be placed not more than 8 feet apart at the circumference.

(*2*) MANHOLE — Suitable access to the bottom of the standpipe shall be provided by means of a manhole not less than $12'' \times 16''$.

(*3*) LADDER — A ladder on the outside, extending to the top, shall be required; and if specified by the engineer one on the inside shall be provided.

(*4*) BALCONY — Around the lower edge of the elevated tank shall be placed a balcony 3 feet wide with a railing $3\frac{1}{2}$ feet high. The floor plate shall be $\frac{1}{4}''$ thick, punched for drainage.

(*5*) PAINT RING — A Z bar with upper flange out and horizontal leg containing drainage holes shall be provided on the outside of the tank near the top.

(*6*) TOWERS — Towers shall have four or six legs beveled radially one in twelve.

(*b*) *Loads*

(*1*) DEAD — Dead load stresses may be estimated in advance, but must be roughly checked from finished design.

(*2*) LIVE — The weight of the water will be taken at 62.5 lbs. per cu.ft. The load per sq.ft. on roof will be estimated as 40 lbs. applied vertically; on walks, 80 lbs.; on ladders, 100 lbs. per lin.ft. vertically or 10 lbs. per lin.ft. horizontally. The rounds of the latter should be designed for a concentrated load of 150 lbs. at center.

(3) WIND — The wind load in lbs. per sq.ft. shall be considered as 30 for a flat surface, 20 on diametrical section of a cylinder, and 15 on said section of a sphere.

(c) *Allowable Stresses in lbs. per sq.in.*

(1) STEEL — The material shall be rivet steel with an ultimate tensile strength of 50,000 for rivets and soft open hearth steel with a tensile strength of 60,000 elsewhere. For such, the allowable stresses will be:

Tension on net section of tank and pipe:	
Plates	12,000
Tension elsewhere	16,000
Compression	$16{,}000 - 90\, l/\rho$

where $\frac{l}{\rho}$ equals ratio of unsupported length to the least radius of gyration.

Shear on shop rivets, pins, and gross section of webs,	12,000
Shear on field rivets and bolts	9,000
Bearing on shop rivets and pins	24,000
Bearing on field rivets and bolts	18,000
Flexure on pins	24,000
Other flexural stresses	16,000

(2) MASONRY — Bearing values, Portland Cement Mortar:

Brickwork	250
Concrete	400
First class sandstone	300
First class limestone	400
First class granite	500

(d) *Design*

The general specifications for bridges in Art. 50 may be used with a few changes. In (2) minimum thickness may be reduced to $\frac{1}{8}''$ for roof and $\frac{1}{4}''$ elsewhere. (4), (6), (8), (11), and (12) will seldom be applicable. In (9) maximum per-

missible slenderness ratios become 175 and 125 for wind and other members respectively. (*15*) should be altered to read as follows:

(*15*) BRACING — Towers shall be braced in each panel of exterior planes and transversely in horizontal planes containing the struts.

(*16*) SHAFE — The pipe and tank shall be cylindrical and the bottom of the latter hemispherical.

(*17*) PAINT RING — The paint ring shall have a section modulus in inches equal to the square of the diameter of the pipe in feet divided by 250.

(*18*) CIRCULAR GIRDER — The circular girder in elevated tanks will be figured for its load as though it were straight.

(*e*) *Details*

A majority of the clauses in Art. 50 apply here also. We note, however, in (*1*) that rivets smaller than $\frac{3}{4}''$ are often employed. We may also add that spacing of rivets near an edge to be calked should not exceed 10 times thickness of plate. Clauses (*6*) concerning rigid members; (*7*) camber, and (*8*) shoes, are evidently inapplicable.

(*f*) *Workmanship*

Referring to (*f*), Art. 50, (*7*) and (*9*) forbidding welds and adjustable members do not apply. For them we may substitute as follows:

CALKING — All plates shall be planed or sheared to the proper bevel before calking. This shall be done from the inside with a round-nosed tool. The use of foreign material is forbidden.

Revise (*2*) to allow material to be punched $\frac{1}{16}''$ larger in the usual manner.

REFERENCE

American Society of Civil Engineers, Vol. 64, p. 526.

Art. 159. Design of a Stand-pipe

See Fig. 159

Let it be required to design a stand-pipe to give a storage of 200,000 gallons over 50 feet above foundations. Allowable stress, 12,000 lbs. per sq.in. on soft open hearth steel. Field rivets, 9000 and 18,000 in shear and bearing respectively. Following the rule developed in Art. 156, we will make about 100 feet high. The required diameter is then 26.1 feet; let us alter the latter to 25 with a height of 105, which approximately fulfills theoretic rule and gives needed capacity.

Assuming two-thirds efficiency, we may write as in Art. 156.

$$t = hd/3200, \text{ or for this case } h = 128t.$$

Assuming plates to be 5 feet in net height, we may compute thickness as follows:

$\frac{1}{4}''$	for the upper 30 feet
$\frac{5}{16}''$	30 to 40 feet down
$\frac{3}{8}''$	40 to 45 "
$\frac{7}{16}''$	45 to 55 "
$\frac{1}{2}''$	55 to 60 "
$\frac{9}{16}''$	60 to 70 "
$\frac{5}{8}''$	70 to 80 "
$\frac{11}{16}''$	80 to 85 "
$\frac{3}{4}''$	85 to 95 "
$\frac{13}{16}''$	95 to 105 "

We will make bottom plate $\frac{5}{8}''$ thick.

The weight of above stand-pipe will be about 200,000 lbs. Taking the wind at 30 lbs. per sq.ft. on a flat surface, equals 20 on the diametrical section of the cylinder, we find its overturning moment to be,

$$20 \times 25 \times 105 \times 52.5 = 2{,}756{,}000 \text{ ft.lbs.}$$

The moment of resistance of the ring at the bottom is $\pi r^2 t = 57{,}500$ in.^{3}

Hence $S = \dfrac{2{,}756{,}000 \cdot 12}{57{,}500} = 575$ lbs. per sq.in. extreme fiber stress. Dead load stress is,

$$\frac{200{,}000}{25 \cdot 12 \cdot \frac{13}{16}\pi} = 260 \text{ lbs. per sq.in.}$$

These are quite small. The stress per lineal inch is (575–260)

$\times \frac{13}{16} = 256$ lbs. If we assume bolts $1\frac{3}{4}''$ in diameter, they have a net area of 1.74 and can safely carry a tension of 21,000 lbs. They should be located at distances not exceeding 21,000/256 equals 82 inches, or say 12 to the circumference.

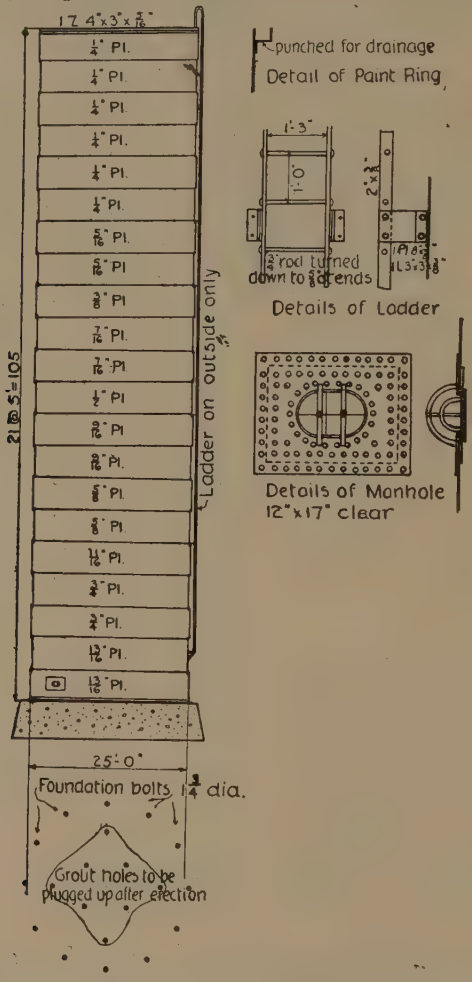


FIG. 159.—General Drawing of Stand-pipe of Problem.
Scale 1″=4′ 0″ and 30′ 0″.

Prob. 159. Design for above stresses a stand-pipe 40 feet in diameter and 80 feet high.

Art. 160. Design of an Elevated Tank

Let us attack the same problem, using an elevated tank with a hemispherical bottom, having the necessary storage capacity in the cylindrical part. This gives us columns for 50 feet. It would probably be more economical to increase diameter somewhat, but for brevity's sake, we desire to avoid refiguring the tank, the upper part of which is now the same as before. We will employ four posts.

Circular Girder

The weight of above tank will be about 80,000 lbs. The total weight of water is 1,940,000 lbs. The load on the circular girder, 17.7 feet between column caps, is then 500,000, giving a maximum shear of 250,000 lbs. and a moment of 13,250,000 in.lbs. A $48'' \times \frac{1}{2}''$ web at 12,000 lbs. per sq.in. is good for a shear of 288,000 lbs. Its equivalent area for moment using the intermediate method is 3.00 sq.in. The net area required is,

$$\frac{3,\ 50{,}000}{47\ \ 16{,}000} = 17.60 \text{ or } 14.60 \text{ sq.in. additional.}$$

Use 1 L $6'' \times 6'' \times \frac{9}{16}''$	Gross area 6.44 sq.in.	Net	5.94
2 Pls. $8'' \times \frac{5}{8}''$	10.00		8.90
Totals	16.44		14.84

Details at the end assume assistance from web of girder and shell of tank in carrying end shear.

Loads on Tower, Fig. 160*a*.

Beveled 1 in 12, radially.

Vertical loads equal one-fourth of (1,940,000 + 80,000) + column, or say 520 kips on each apex.

For the purpose of obtaining the wind pressure, we shall assume a hemispherical roof. The total wind load will be,

$$20 \times 55 \times 25 + 15 \times .7854 \times 25 \times 25 = 35{,}000 \text{ lbs.},$$

which will be applied 77.5 feet above foundations or 27.5 above column caps. The wind load half way up on the tower will be taken as 4 kips at each corner with 2 kips top and bottom.

There are two cases:

I. Wind blowing diagonally with respect to towers.

II. Wind blowing parallel to one set of struts.

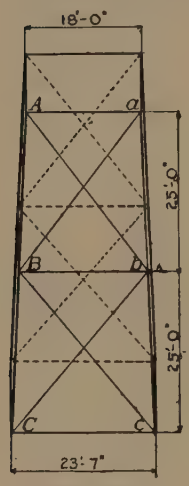


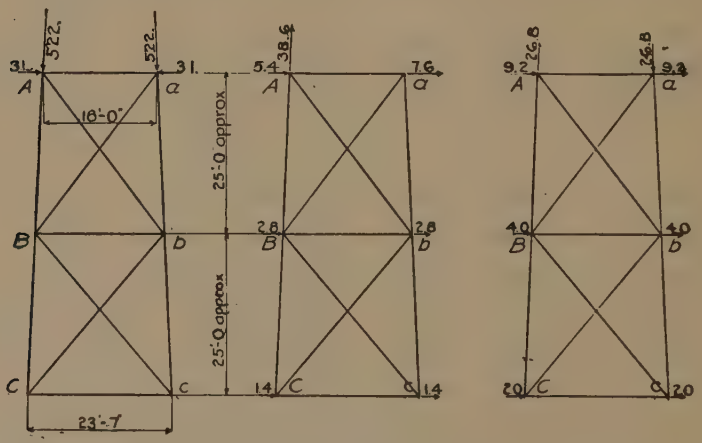
FIG. 160*a*.
The Tower.

In either case horizontal reactions are assumed to be divided equally between the four posts, that is, the load from the tank will be divided 8.8 kips to each. The total at the top will then be 10.8; at middle, 4; and at bottom, 2 kips. For Case I, we resolve into the two rectangular components of 7.6, 2.8, and 1.4 kips, parallel to the horizontal struts.

The moments of the wind loads about top of columns is,

$$35 \times 27.5 = 962 \text{ kip ft.}$$

For Case I, the neutral axis is a diagonal line through two posts of the tower. This moment is carried equally by the two other columns, each at a distance of 12.5 feet from neutral axis. Dividing above moment by two and then by this distance, we obtain for the reaction,



Vertical Loads. Case I. Case II.

FIG. 160*b*.—Developed Views of One Side of Tower under Various Loads.

38.5 kips, up on one side and down on the other. For Case II, the neutral axis is perpendicular to the wind and passes through the center of symmetry of the tower. By a similar method we obtain a reaction upward on the leeward columns, and downward on the windward side of 26.7 kips. Each of these, together with the horizontal loads, may in turn be resolved into three components parallel to post and its two top struts, giving results shown in Fig. 160*b*. We may now compute as follows:

	AB.		*BC.*		*Aa.*	*Bb.*	*Cc.*	*aB.*	*bC'*
Vertical.....	522C		522C		30.8C	0	30.8T	0	0
Wind, Case I .	39T	54C	54T	72C	7.6C	13.8C	7.8C	19.7T	24.0T
Wind, Case II.	27T	49C	49T	74C	9.2C	19.6C	13.5C	28.0T	33.8T
Total.....	576C		596C		40.0C	19.6C	30.8T 13.5C	28.0T	33.8T

Note that top struts are imaginary, as they would actually interfere with tank.

Posts

Assuming that allowable stress will be 14,000 lbs. per sq.in., we will need 42.5 sq.in. in posts. This is too large for rolled channels, so we will use built. We will try 2 Pls. $20'' \times \frac{5}{8}''$ and 4 L's $4'' \times 4'' \times \frac{5}{8}''$, the latter being placed $20\frac{1}{4}''$ back to back along the plate and far enough apart in the other direction to render radii of gyration alike. Combined area is 43.48 sq.in., radius of gyration, 7.31", and allowable total stress $43.48\left(16{,}000 - 90\frac{300}{7.31}\right) = 535$ kips. This is not quite enough but obviously if we increase the thickness of plates and angles to $\frac{11}{16}''$, it will be just right.

Struts

2 [s 8"@13.75# have a combined area of 8.08 sq.in. and a capacity in compression for a length of 23' 7" of,

$$8.08\left(16{,}000 - 90\frac{283}{2.98}\right) = 60 \text{ kips, which will suffice for all.}$$

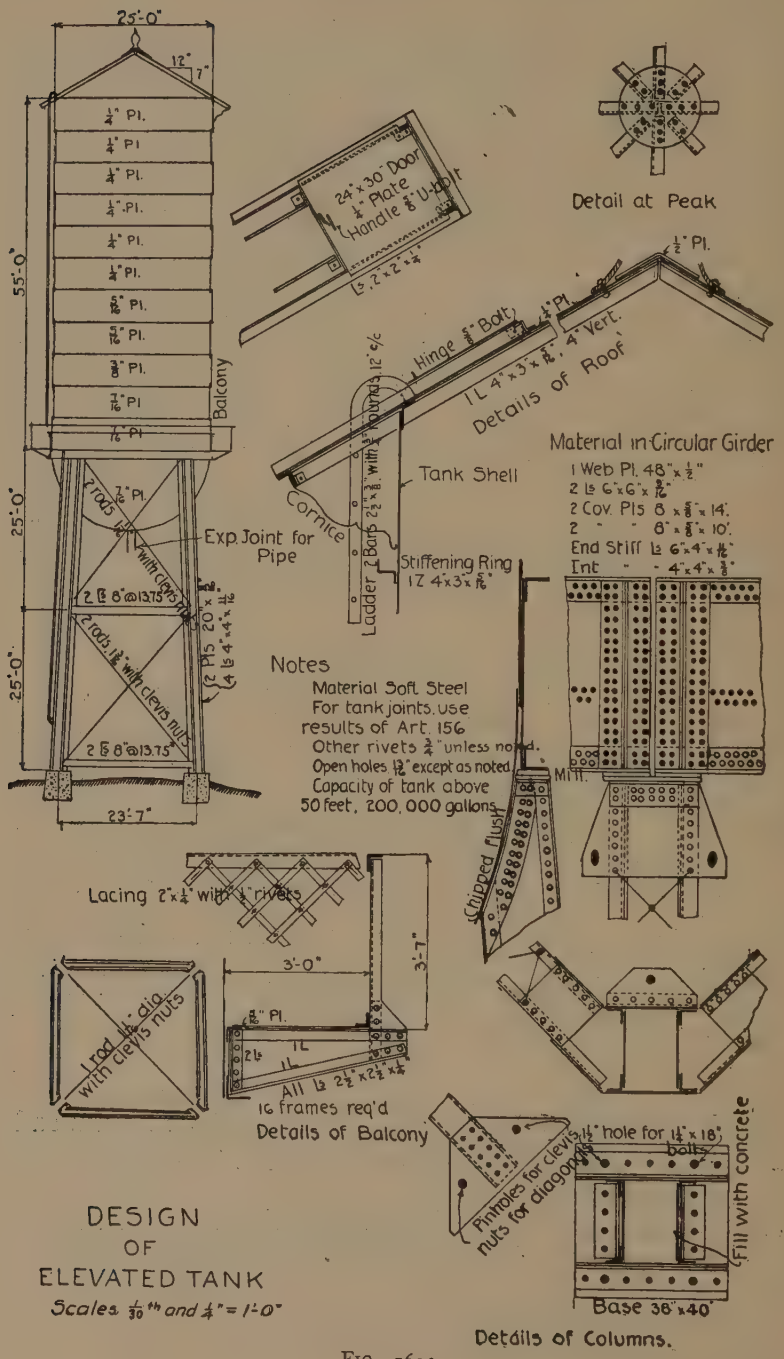

25'-0"
12"
7"
¼" Pl.
¼" Pl
¼" Pl.
¼" Pl.
¼" Pl.
¼" Pl.
⅝" Pl.
⅝" Pl.
⅜" Pl.
7/16" Pl
7/16" Pl
55'-0"
Balcony
25'-0"
25'-0"
7/16" Pl.
2 rods 1¼" with clevis nuts
Exp. Joint for Pipe
2 rods 1¼" with clevis nuts
2 [8" @ 13.75
2 Pls 20" x ⅜"
4 L 4" x 4" x ½"
2 [8" @ 13.75
23'-7"
24" x 30" Door
½" Plate
Handle ⅝" U-bolt
L 2" x 2" x ¼"
Detail at Peak
½" Pl.
Hinge ⅝" Bolt
¼" Pl.
1 L 4" x 3" x ⅝", 4" Vert.
Details of Roof
Material in Circular Girder
1 Web Pl. 48" x ½"
2 L 6" x 6" x ⅝"
2 Cov. Pls 8" x ⅝" x 14'.
2 " " 8" x ⅝" x 10'.
End Stiff L 6" x 4" x ⅝"
Int " 4" x 4" x ⅜"
Cornice
Tank Shell
Ladder 2 Bars 2½" x ⅜" with ¾" rounds 12" c/c
Stiffening Ring
12 4" x 3" x ⅜"
Notes
Material Soft Steel
For tank joints use results of Art. 156
Other rivets ¾" unless noted.
Open holes ⅞" except as noted.
Capacity of tank above 50 feet, 200,000 gallons
Mill
Stripped flush
Lacing 2" x ¼" with ¾" rivets
3'-0"
3'-7"
1 rod ⅞" dia. with clevis nuts
⅝" Pl.
1L
1L
All L 2½" x 2½" x ¼"
16 frames req'd
Details of Balcony
Pinholes for clevis nuts for diagonals
1½" hole for 1¼" x 18" bolts
Fill with concrete
Base 36" x 40"
Details of Columns.
DESIGN
OF
ELEVATED TANK
Scales 1/30th and ¼" = 1'-0"

FIG. 160c.

Diagonals

aB Area required 28.0/16.0 = 1.75, use 2 round rods, 1–$\frac{1}{16}$″ dia.
bC Area required 33.8/16.0 = 2.10, use 2 round rods, 1–$\frac{3}{16}$″ dia.
To be upset at ends.

Base

Area of base required for concrete footings is 596/0.4 = 1490 sq.in. Provision must be made for an uplift of about 30 kips; use 4 1$\frac{1}{4}$″ bolts, giving ample security.

END VOLUME II.

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